APPENDIX A

ENGINEERING APPENDIX

This page intentionally left blank

TABLE OF CONTENTS

ANNEXES

LIST OF TABLES

LIST OF FIGURES

A.0 ENGINEERING DESIGN APPENDIX

The Engineering Appendix of the Project Implementation Report (PIR) provides a comprehensive record of the technical support provided by the USACE Jacksonville District Engineering Division to the Central Everglades Planning Project, with technical information and analyses provided by the following engineering disciplines: Civil, Cost, Electrical, Geotechnical, Mechanical, Structural and Water Resources. The main Engineering Appendix, which is organized by technical discipline within each geographic sub-region, includes the following general information: an overview of the features of the CEPP Recommended Plan, overview status of Engineering design activities and analyses, discussion of general construction procedures, overview of preliminary civil site design information; overview of geotechnical considerations and analyses; overview of hydrologic and hydraulic design and analyses; documentation of the hydrologic modeling, and a summary of value engineering analyses. For the summary of costs, cost considerations and assumptions, refer to Appendix B – Cost Engineering. Consistent with the formulation approach, the geographic sub-regions are defined as: 1) North of the Redline (Storage and Treatment); 2) South of the Redline (Diversion and Conveyance) and 3) Blue Green Yellow line (Distribution, Conveyance and Seepage Management).

A.1 CENTRAL EVERGLADES PLANNING PROJECT

The study area for the Central Everglades Planning Project (CEPP) encompasses the Northern Estuaries (St. Lucie River and Indian River Lagoon and the Caloosahatchee River and Estuary), Lake Okeechobee, a portion of the Everglades Agricultural Area (EAA), the Water Conservation Areas (WCAs), Everglades National Park (ENP), the Southern Estuaries (Florida Bay and Biscayne Bay), and the Lower East Coast (LEC). The purpose of CEPP is to improve the quantity, quality, timing and distribution of water flows to the Central Everglades. Existing Conditions are summarized in **Section 2.0** (Existing and Future Conditions) of the main PIR Report and Environmental Impact Statement (EIS). Greater detail is further described and provided in Appendix C.1.

A.2 RECOMMENDED PLAN

The recommended plan will provide approximately 210,000 ac-ft per year of additional water flow to the Everglades (measured at the Redline) by redirecting water which is currently being discharged to tide via the St. Lucie and Caloosahatchee Estuaries south to the existing EAA canals. The EAA Miami Canal and North New River Canal will convey the redirected flows to the recommended plan FEB storage feature, which will attenuate flow rates prior to water quality treatment using available, off-peak capacity of the state-operated STA-2 and STA-3/4. Following water quality treatment, this additional flow quantity will be re-distributed as inflows to WCA 2A and WCA 3A, and the recommended plan features will modify the quantity, quality, timing, and spatial distribution of flows into and through WCA 3A, WCA 3B, and ENP to Florida Bay in order to meet the project objectives. This plan would be accomplished by a combination of modifications to the existing Central and South Florida project components, construction of additional components, and modifications to current approved water control manuals. Several proposed or existing levees, canals, and culverts, and pump stations would be constructed, modified, or removed to improve the flow of water through the system as the first increment of CEPP.

The recommended plan was refined from Alternative 4 to Alternative 4R2 through the formulation process (refer to **Section 4.6** of the PIR main report). The recommended plan elements were further refined based on engineering analysis, preliminary design recommendations, costs described below and in detail in this **Appendix A – Engineering**. The limited engineering and design analysis does not affect the plan formulation, as the cost changes and project refinements would similarly apply to all alternatives. Similarly, the total benefits derived by the plan are not anticipated to significantly change based on these engineering refinements. The recommended plan includes features in three major studied areas: North of the Redline, South of the Redline, and along Blue Green Yellowline. For general graphical depictions of the recommended plan features, please refer to **Section 6** of the PIR main report. This appendix includes additional figures to describe the engineering refinements to the recommended plan, where appropriate.

Features in the EAA (North of the Redline) include construction of the 14,000 acre A-2 Flow Equalization Basin (FEB) (L-624 perimeter levee and L-625 interior levee; C-624, C-624E, C-626 internal distribution channels; S-623, S-624, S-628 inlet structures; S-625 outlet structures, and C-625E, C-625W canals and channels connecting the FEB to the Miami Canal). Operation of the A-2 FEB would be integrated with the operation of the A-1 FEB, a state-funded and state-constructed FEB.

Conveyance features in WCA 2A and northern WCA 3A (South of the Redline) include: S-620, a gated culvert to deliver water from the L-6 Canal to the remnant L-5 Canal; S-622, a new gated spillway to deliver water from the remnant L-5 canal to the western L-5 canal (during L-6 diversion operations); S-621, a new gated spillway to deliver water from STA 3/4 to the S-7 pump station during peak discharge events (eastern flow route is not typically used during normal operations), including L-6 diversion operations; conveyance improvements to approximately 13.6 miles of the L-5 Canal; degrade approximately 2.9 miles of the southern L-4 Levee along the northwest boundary of WCA-3A; S-630, a new 360 cfs pump station to move water within the L-4 Canal to maintain water supply deliveries to maintain existing functionality of STA-5 and STA-6 and maintain water supply to existing legal users, including the Seminole Tribe of Florida; S-8A new gated culverts to deliver water from the Miami Canal (downstream of S-8, which pulls water from the L-5 Canal) to the L-4 Canal; and backfill approximately 13.5 miles of the Miami Canal and include constructed tree island mounds, between a point approximately 1.5 miles south of the S-8 pump station and Interstate Highway I-75.

Additional conveyance features that would be located in southern WCA 3A, WCA 3B, and the northern edge of ENP (Blue Green line) include: S-333N, a 1,150 cfs gated spillway adjacent to S-333; S-631, a 500 cfs gated culvert in L-67A Levee and an associated 6,000 foot gap in the L-67C Levee; a flowway through the western end of WCA 3B (S-632 and S-633 2 gated culverts in L-67A Levee; removal of approximately 8 miles of L-67C Levee; removal of approximately 4.3 miles of L-29 Levee; construct L-67D a new approximately 8.5 mile levee); S-355W, a gated spillway in the L-29 Canal to maintain water deliveries in the L-29 Canal to the eastern Modified Water Deliveries (MWD) 1-mile bridge and maintain western access to the L-29 Levee; remove approximately 5.5 miles of the L-67 Extension Levee; and remove approximately 6 miles of Old Tamiami Trail between the Everglades National Park (ENP) Tram Road and the L-67 Extension Levee. Work in this area also includes removal of spoil along the western L-67A canal in the vicinity of the new control structures and removal of vegetation along WCA-3B agricultural ditches.

Features primarily for seepage management (Yellowline), which are required to mitigate for increased seepage resultant from the Blue Green line features include: S-356, a new 1,000 cfs pump station to replace the existing temporary S-356 pump station; and an approximately 4.2 mile long, 35 feet deep tapering (potential variable seepage wall depths) seepage barrier cutoff wall along the L-31N Levee, just south of Tamiami Trail and east of the ENP Northeast Shark River Slough (NESRS).

To address quality, quantity, timing and distribution of the water through the CEPP project various types of infrastructure were considered during the formulation process such as: Stormwater Treatment Areas (STAs), a Flow Equalization Basin (FEB), deep storage reservoir, spreader canals, pumps, canal backfilling and canal plugs, levee removal and levee gaps, culverts/gated structures, seepage barrier walls, seepage control pumps, hydraulic ridge detention areas, and step down levees.

To meet the overall objectives of CEPP to deliver additional water to the Everglades system, the project features detailed in **Table A-1** were selected as components of the recommended plan, or Recommended Plan to best achieve the goals for this project based on cost effective benefits.

A.2.1 PROJECT FEATURES

TABLE A-1. SUMMARY OF RECOMMENDED PLAN FEATURES

Structure/Feature Number	Structure/Feature Type	Design Capacity (cfs)	Location	Tech Specs & Notes
NORTH OF THE REDLINE – STORAGE AND TREATMENT FLOW EQUALIZATION BASIN (FEB) – A 2				
S-623 (DS-8)	Gated Spillway	3700	On STA 3/4 Supply Canal	Delivers water from Miami Canal to existing G-372. When closed, FEB outflows are isolated from the Miami Canal for delivery to STA-3/4.
S-624 (DS-5)	Gated Sag Culvert (FEB inflow structure)	1550	On STA 3 / 4 Supply Canal	Receives water from G-372 via STA 3 / 4 Supply Canal and delivers to C-624 canal.
S-625 (DS-7)	Gated Culverts (FEB discharge structure)	1550	Discharge structure in FEB perimeter levee L-624	Delivers water to FEB outflow canal
S-626 (PS-1)	Seepage Pump Station	500	West side of seepage canal, C-626	Delivers seepage back into the FEB outflow canal C-625W
S-627 (CS-4)	Emergency Overflow weir	445	Between A-2 and A-1 FEB, just north of S-628	445 cfs for 100-yr 24 hr (per DCM-2)
S-628 (DS-9)	Gated Culvert FEB intake/ discharge structure	930	Between A-2 and A-1 FEB	Delivers water in both directions between A-2 and A-1 FEB
L-624	Levee		FEB Perimeter Levee	~ 20 miles, 11.3 height, 14ft width, 3:1 side slopes
L-625	Levee		FEB interior inflow canal levee	~ 4 miles, 11.3 height, 12ft width, 3:1 side slopes
C-624	Inflow Canal	1550	West side interior of FEB	~ 4 miles
C-624E	Spreader Canal		Northern boundary of FEB	~ 4 miles
C-625E	Collection Canal	400	FEB interior collection canal along southern perimeter	Existing seepage canal for STA 3 /4 Supply Canal will be repurposed and used to supplement FEB sheetflow to S-625 during normal operating conditions; C-625E provides primary conveyance to S-625 when no sustained pool depth (i.e., only sheet flow)
C-625W	Outflow Canal	1550	FEB exterior outflow; between S-625 and G-372 headwater	FEB outflow canal is the extended seepage canal for the STA 3 / 4 Supply Canal
C-626	Seepage Canal	400	West and northern exterior perimeter of FEB	~ 11 miles
SOUTH OF THE REDLINE – DIVERSION & CONVEYANCE				
S-620 (CS-1)	Gated Culvert	500	In L-6 Canal	Delivers water from L-6 canal to L-5 canal

TABLE A-1. SUMMARY OF RECOMMENDED PLAN FEATURES CONT'D				
Structure/Feature Number	**Structure/Feature Type**	**Design Capacity (cfs)**	**Location**	**Tech Specs & Notes**
SOUTH OF THE REDLINE CONT'D – DIVERSION & CONVEYANCE				
S-621 (CS-2)	Gated Spillway	2500	On STA 3 / 4 Outflow Canal	Closed to direct STA 3 /4 discharges to western L-5 Canal during normal operations; controls water from STA 3 /4 to the S-7 pump station during peak events.
S-622 (CS-3)	Gated Spillway	500	In L-5 Canal	Delivers water from east to west in L-5 canal
New (S-8A)	Gated Culverts w/canal	3080 & 1020	In Miami and L-4 Canal	Delivers water from the Miami Canal west to L-4 (3120 cfs) and to the remaining Miami Canal segment 1040 cfs). S-8 delivers water from the L-5 Canal to the Miami Canal, upstream of S-8A. Potential design modifications to the existing S-8/G-404 complex will be assessed during PED.
S-630	Pump Station	360	In L-4 Canal	Delivers water from L-4 Canal west to maintain existing water supply deliveries
	Levee Removal		L-4 Interior Levee	Remove ~2.9 miles, 6ft ht,, 10ft width, 2.5:1 side slopes
	Canal Backfill		Miami Canal	Remove ~ 13.5 miles
	Tree Islands Mounds		Miami Canal	Create habitat and promote sheetflow in WCA-3A
	Canal	500	Remnant L-5 Canal east	Enlarging canal between S-621 and S-622
	Canal	3000	L-5 Canal west	Enlarging canal between S-622 and S-8
BLUE GREEN YELLOW LINE – DISTRIBUTION, CONVEYANCE & SEEPAGE MANAGEMENT				
S-333 (N)	Gated Spillway w/new canal	1150	Just north of existing S-333	Delivers water from L-67A Canal to L-29 Canal
New S-356	Pump Station	1000	In vicinity of existing temporary S-356	Provides seepage management for WCA 3B and NESRS stages
S-631	Gated Culvert	500	In L-67A Levee	Delivers water from WCA 3A to 3B, east of L-67D Levee
S-632	Gated Culvert	500	In L-67A Levee	Delivers water from WCA 3A to 3B, west of L-67D Levee
S-633	Gated Culvert	500	In L-67A Levee	Delivers water from WCA 3A to 3B, west of L-67D Levee
	Levee Removal Gap		L-67C Levee	~ 6000 ft gap corresponding to S-631, 5ft ht, 10ft width, 3:1

TABLE A-1. SUMMARY OF RECOMMENDED PLAN FEATURES CONT'D				
Structure/Feature Number	**Structure/Feature Type**	**Design Capacity (cfs)**	**Location**	**Tech Specs & Notes**
BLUE GREEN YELLOW LINE CONT'D – DISTRIBUTION, CONVEYANCE & SEEPAGE MANAGEMENT				
L-67D	New Levee		In WCA 3B	~ 8.5 miles connects from L-67A to L-29, 6 ft height, 14 ft crest width, 3:1 side slopes
	Levee Removal		L-67C Levee	~ 8 miles complete removal from New Levee (L-67D)south to intersection of L-67A/L-67C; L-67C canal is not backfilled
S-355W	Gated Spillway	1230	In L29 Canal, east of L-67D Levee terminus and 2.6 mile bridge	Maintains water deliveries in L-29 Canal to 1-mile bridge & maintains access for Tigertail Camp to Tamiami Trail.
	Levee Removal		L-29 levee	~ 4.3 miles removal east of ValuJet monument to L-67D Levee intersection with L-29 Levee. 10ft ht, 10ft width, 3:1
	Road Removal		Old Tamiami Trail (from L-67 Ext west to ENP Tram Rd)	~ 6 miles of Old roadway removal , 5 ft height, 30 ft width, 2:1 side slopes
	Levee Removal and Canal Backfill		L-67 Ext levee and Canal	~ 5.5 miles complete removal of L-67 Ext; 8ft height, 10ft width, 3:1 side slopes
	Seepage Barrier Cutoff Wall		In L-31N levee just south of Tamiami Trail	~4.2 miles of 3ft wide, 35 ft deep, Soil Cement Bentonite (SCB) Wall
S-346	2-72" metal culvert w/Flash Board Removal	165	In Old Tamiami Trail	Anticipate removal if ~5.5 miles of L-67 Ext removed

Key: cfs = cubic feet per second

A.2.2 Pre-Recommended Plan Design

Due to an expedited schedule, absence of site specific data and limited data, design for alternative development employed best professional judgment and prior knowledge of existing CERP components. The assumptions and limited design are captured in latter sections of this Appendix. As documented in this appendix many traditional design analyses are delayed to future design phase. The assumptions and limited design to date contributed to the development of the Final Array of Alternatives. For a description of the Final Array of Alternatives, see main PIR document **Section 3** Formulation of Alternative Plans and **Section 4** Evaluation and Comparison of Alternative Plans.

A.2.3 Cost Estimates

Refer to PIR main report, Appendix B – Cost Engineering for cost development and methods to include the CEPP Recommended Plan cost.

A.3 STATUS OF ENGINEERING DESIGN ACTIVITIES AND ANALYSES

A.3.1 Level of Design Efforts

Design Engineering Regulation, ER 1110-2-1150, Engineering and Design for Civil Works Projects provides guidance for Feasibility level design to accompany decision documents. Early during CEPP project scoping, risks were identified that accompanied the expedited pilot planning process. The risks were presented in a project risk register. Those pertaining to Engineering were: ENG-01 Limited Data for Engineering Design; ENG-02 Design Details accomplished and ENG-13 Levee Safety Analysis. With the Risks identified and Decision Point (DP) 1 meeting, it was marked that due to the expedited schedule to execute and the limited ability to acquire site specific data, traditional analyses typical for feasibility level design would not be accomplished for inclusion in this project PIR report. The team identified work that would be deferred to preconstruction engineering and design phase (PED). The up-front project risks recognized the potential for these design activities to significantly affect project costs. Due to the limited design, it is expected that higher risk based contingencies would be generated yet Cost certification would still be achieved. This is in accordance with additional guidance from Engineering Construction Bulletin (ECB) 2012-18.

A.3.2 Recommendation for Design Completion

Features of the Recommended Plan have been identified according to available data, historic information, and best engineering judgment. All project components will be optimized during PED phase for cost efficiency and performance, incorporating updated data and information as it becomes available. Design completion recommendations are provided by geographic region and discipline specific areas.

A.4 GENERAL CONSTRUCTION PROCEDURES DISCUSSION

A.4.1 General Construction Recommendations

It is envisioned that the project will be constructed using conventional means and methods. The project features were scoped by project areas and conceptually placed in contracts that maximize opportunities to realize benefits with clean water already in the existing system. The features/contracts capitalize on use of onsite material, reduce multiple handling scenarios, and to maintain flood control operations and level of service provided by existing features. Other schedule and construction assumptions included that all engineering design work would be completed by USACE with in-house resources. Beginning with investigative information gathering, multiple contracts would be awarded every year based on construction durations estimated from existing similar USACE construction projects. Adaptive Management will help with future development of the implementation and sequencing. During PED, detailed analyses, subsurface investigations and site investigations will be conducted to prepare construction documents.

A.4.1.1 North of Redline

It is assumed that the A-1 FEB will be completed in FY 2018 and operated at least 5 years prior to any construction of the CEPP A-2 FEB. The A-2 FEB would not initiate construction until the Blue Green Yellowline Seepage Barrier is in place. There are multiple structures, canals and levees with subsequent access and electrical works. It is generally assumed that construction for the A-2 FEB would start north east ward and proceed west and south. During PED construction sequencing will be further investigated and defined for USACE contracts.

A.4.1.2 South of Redline

It is assumed that the work in the area would need to be sequenced together based on earthwork dependencies. The L-6 diversion structures with incidental canal improvement may be performed independent of L-5 canal conveyance improvements. The L-4 degrade and S-630 construction should be performed in the same contract concurrently. S-8 modifications should be completed to permit the diversion of L-6 flows and must maintain flood control operation capability during S-8 modifications. Miami Canal backfilling is primarily dependent upon material from the L-5 canal conveyance improvements with other material coming from the L-4 levee degrade, new S-8A canal (that connects the L-4 and Miami Canal) and the adjacent spoil mounds located along the Miami Canal north of Interstate 75 (I-75). These two features should be performed concurrently in a contract. There are multiple structures, canals and levees with subsequent access and electrical works. It is generally assumed that construction for the Miami Canal backfilling would start north and proceed south to S-339 and eventually to I-75. Conceptual construction impacts acres are provided in the main PIR document. During PED construction requirements and sequencing will be further investigated and defined for USACE contracts. See Appendix A, Annex C-2 for conceptual degrade and backfill plates.

A.4.1.2.1 Miami Canal Tree Island Construction/Planting

Approximately 13.5 miles of Miami Canal will be backfilled to bedrock from about a mile south of S-8 to S-339 and to about one foot above bedrock from S-339 to I-75 so that the entire length of the backfilled canal template will be ~1.5 below the peat surface. All spoil mounds on the east and west side of the Miami Canal will be removed from S-8 to S-339. From S-339 to I-75 all spoil mounds will be removed except for 22 FWC enhanced spoil mounds identified by FWC as the highest priority. In addition to the backfilling, CEPP will construct and create (14) tree islands approximately every mile along the entire reach of the Miami canal (S-8 to I-75) where historic tree islands once existed. The remaining FWC spoil mounds will be incorporated into the constructed tree island that will be constructed along the ridges of the historic ridge and slough landscape to use as potential tree island generators.

Purpose of Tree Islands
The constructed tree islands are intended to block flow down the backfilled canal by having a profile across the landscape that varies, or undulates in elevation. The longitudinal cross section of this series of tree islands varies from marsh grade to ~1.5’ above marsh grade. This undulated elevation will provide somewhat natural slopes for vegetation and wildlife, provide higher habitat for diverse plant and animal species that require such habitat, and provide low elevation slough areas between each island to promote natural water flow paths through the Everglades.

Recommendations for the composition, design and construction of the tree islands in full detail and reference project photos with graphics are provided in Appendix A, Annex C-2. A summary of the ecological recommendation is provided below. Constructed tree island design details will be determined during CEPP PED phase. CEPP PED discussions regarding Miami Canal backfilling and tree island construction/planting will involve coordination with appropriate science team members with expertise in these topics to accomplish the restoration vision and intent of CEPP’s backfilling and tree island construction. Scientists included in the PED effort will bring information from Florida Fish and Wildlife

Commission and Loxahatchee Impoundment Landscape Assessment (LILA) tree island design and planting projects, and other relevant efforts.

Islands from S-8 to S-339

The constructed tree islands from S-8 to S-339 will range from 280' to 210' in total width and overall length from 210' to 310'. The flat area should be 1.5' above marsh grade, with a maximum 18:1 transition to marsh grade.

Islands from S-339 to I-75

The constructed tree islands from S-339 to I-75 are adjusted slightly from that described above to blend constructed islands with the remaining portions of the FWC planted tree islands. These constructed islands range from approximately 500' to 1500' long and up to 210' in width. All areas with a FWC Island will have at its northern end a flat portion, raised 1.5' above marsh grade, that abuts and transitions into the adjacent, existing FWC Island at a maximum 18:1 down or up to marsh grade. For specific details on the possible configurations see Appendix A, Annex C-2.

Construction of Tree Islands

A summary of the ecological recommendation is provided below. Miami Canal will be filled to bedrock level using local compacted fill material that excludes branches, trunks, or organic material. The layer of fill from the top of the bedrock to the elevation of the tree island will include a wide range of grain and rock sizes as well as non-uniform, randomly placed, non-mulched branches and trunks from Miami Canal spoil material after consultation with FDEP during the PED phase. This would provide the needed porosity throughout the tree island to the bedrock level. The goal is to have enough porosity for plant roots to be able to reach water through the island material which is an essential need for tree island creation. The end result should be rough terrain that is "difficult to walk on." See Appendix A, Annex C-2, LILA construction photos for examples. Size of rock and volume will need to be specified. Organic muck should be spread 12" to 18" thick over the planting areas (holes) to help plants have room to establish. The top layer of organic material is critical in the planting holes, where the plantings will be expected to root and grow.

Once islands are created, planting holes will be created and filled them with organic material which provides a rooting medium for the native plants and promote survival in harsh conditions without irrigation. The holes should be at least 1-ft diameter per tree. It is suggested that the hole depths can be a randomly distributed mixture of 1/3 each at 1', 2', 3' depths. Details can be determined during PED based on contracting capabilities. Contracting options will be fully explored during PED to accomplish Miami Canal backfilling, tree island creation and plantings.

Tree Island Planting

Local plant sources are recommended for planting to maintain Everglades' genetic consistency among planted seedlings. All proposed plantings are assuming 3-gallon size plants. Immediately after planting, individual plants will be protected by ~3-ft metal enclosure, secured by metal stakes, to deter herbivores while the plants are becoming established. A diverse array of species will be planted, including trees, shrubs, and herbaceous species that are appropriate for these tree islands. The array will include a mix of faster-growing, desirable native species that will help to quickly create an environment on the constructed tree islands that is conducive for restoration, i.e., that will shade out weedy species and protect the organic layer on the island, will quickly develop root systems, and that will attract wildlife to the islands. Other desirable species that may grow more slowly will also be planted to result in the appropriate species composition for restored tree islands in this area. In addition, species will be chosen that are fire tolerant for the outer edges of the constructed tree islands to buffer the inner island area from wildfires. It is expected that the islands will accrue additional native species over time by natural recruitment.

It is recommended that construction of the tree islands not take place in the wet season, but recommended that planting be done at beginning of the wet season to attempt to avoid irrigation. However, design and construction sequencing to include planting will be detailed in PED phase. Refer to Appendix A, Annex C-2, Civil Plates for full recommendation details.

A.4.1.3 Blue/Green/Yellowline

It is assumed the Tamiami Trail western 2.6 mile bridge and appropriate road raising are completed by FY 2022 under the Department of Interior (DOI) Tamiami Trail Modifications: Next Steps project. Passive control features were screened out during the CEPP plan formulation process and will not be further considered during future CEPP implementation. Active control structures, such as the gated culverts along L-67A included in the CEPP Recommended Plan, are required to most effectively address: adaptive management flexibility and system uncertainties (the WCA-3A regulation schedule varies seasonally, whereas passive weir elevations are most likely predetermined and static); water quality considerations and constraints; T&E species considerations within WCA-3A and ENP, including flexibility for management of recession/ascension rate targets; and surface water velocity considerations within the flowway. Further, the CEPP modeling and preliminary DPOM recognize that the only anticipated operational constraint for the proposed controllable L-67A structures within the Blue Shanty Flowway (S-632 and S-633) would be the 9.7 feet NGVD maximum stage elevation for the L-29 Canal based on the planned DOI TTNS Tamiami Trail roadway modifications, and this same constraint would equally apply under a passive weir scenario.

It is assumed that CEPP work in the area would need to be sequenced together based on earthwork dependencies. The L-67A structure S-631 and designated L-67C gapping would be performed first as an adaptive management strategy. The new S-333N, S-355W and S-356 structures may be completed as separate contracts independent of each other and other work in the area. They are not dependent upon any other new feature but must consider the existing facilities currently or planned to be in use in the construction vicinity. The existing S-356 is a temporary pump station which could be replaced as the first contract in this area, to maintain seepage mitigation associated with operation of the existing temporary S-356 pump station. The remaining L-67A structures (S-632 and S-633), designated L-67C levee removal with associated spoil removal westward of L-67A canal and L-67 extension levee removal may be performed concurrently with construction of new WCA 3B levee (L-67D). Including L-67 extension levee removal with this work assists with utilization of onsite material for the new L-67D construction. The new levee (L-67D) will require a variance from USACE mandatory vegetation management standards found in Engineering Technical Letter (ETL) 1110-2-571, Vegetation Standards for Levees and Floodwalls Safety Guidance for vegetation management zones. This variance will maintain 15 foot clearance on each side of L-67D of woody vegetation but allow marshy vegetation within the zone. The next sequenced contract in this area would be removal of the L-29 levee since it is contingent upon total L-67D levee and the S-355W being in place. The L-29 levee removal would proceed starting east of the ValuJet monument and progress east to coincide with L-67D terminus. The material from the L-29 levee removal would be used in the L-67 extension canal backfill. It is assumed that the last works in this area, which include the new L-31N Seepage barrier cutoff wall, Old Tamiami Trail removal and L-67 extension canal backfill, are each independent contracts. L-67 extension canal backfill will not be sequenced prior to the removal of the Old Tamiami Trail, in order to maintain the conveyance capability of the S-12s to discharge excess water from WCA-3A. There are remaining uncertainties about the effectiveness of the CEPP Recommended Plan seepage cutoff wall in maintaining desired stages in marshes of ENP while maintaining flood protection and canal stages to the east without limiting water availability to water users and Biscayne Bay. Therefore, additional analysis of the CEPP seepage cutoff wall will be conducted as an early phase in PED. See Section 6.10.2.1, the Engineering Appendix (Appendix A), the analyses required by WRDA 2000 (Annex B), and the CEPP Adaptive Management Plan (Annex D Part 1) for detail. The A-2 FEB construction could initiate immediately following or with the L-31N Seepage barrier wall. There are multiple canals, ditches and levees with subsequent access and electrical works that will require coordination to minimize cultural resource and wetland impacts. Conceptual construction impacts acres are provided in the main document. During detail design phase construction sequencing will be further investigated and defined for USACE contracts. See Appendix A, Annex C-2 for conceptual plates.

A.5 NORTH OF THE REDLINE – FLOW EQUALIZATION BASIN

A.5.1 CIVIL - SITE DESIGN

Features identified in the Recommended Plan have been designed to the level of detail necessary to provide cost estimates. Best professional judgment and previous project design knowledge for EAA and the SFWMD A-1 FEB were used during plan formulation alternative development and design efforts. Components north of the redline have been identified according to available data, historic information, and best engineering judgment. All project components will be optimized during PED phase for cost efficiency and performance, incorporating updated data and information as it becomes available.

All levees in the Recommended Plan have a crown of 14 feet with one on three side slopes. The levees will be seeded to prevent erosion. All perimeter levees will have a 15 foot clear zone at the toe as required by EM 1110-2-1913, Design and Construction of Levees. For the internal levee, a variance will be investigated during PED. Upon completion of construction the levees will be entered into the National Levee Data Base for regular inspections as required by P.L. 84-99 to be part of the Federal Emergency Management System.

A.5.1.1 General Status of Completed and Non-Executed Efforts

The following civil site project efforts remain either incomplete or were not initiated:
evaluation of alignments,
site grading,
aesthetics,
relocation of facilities,
required improvements on lands to enable proper construction of components and disposal of material,
requirements of lands for construction,
operation and maintenance of the project,
identification of methods for accomplishing relocations to include appropriate lands,
site selection and project development, and
design with respect to recent Levee Safety criteria.

These analyses will be completed in PED.

A.5.1.2 Surveying Mapping Geospatial data

During PED phase site specific surveys of the features, utilities to be relocated and internal canals to be filled shall be done for the design. There is no survey information available for this area to utilize for civil site design. All survey will be in 1988 NAVD with a 1929 NGVD correction as required for the Water Control Plan. See Appendix A, Annex C-1 for data points.

A.5.1.3 Access

Access to this area is from US 27 utilizing the A-1 FEB access road that connects the northeast corner of the A-1 to the recreation area by culverts. There is an existing east-west road, “Central Agricultural Road”, that could provide direct access to the A-2 footprint, but this is projected to be degraded for the A-1 project. If new haul roads are needed, they will have to divert from the nearest public road and will be forward placed limestone displacing the underlying peat materials. Access within the A-2 FEB footprint will be accomplished using and improving existing local levees and roads.

A.5.1.4 Material Balance and Disposal

Cut and fill quantities will be completed during PED phase to balance the design as much as possible. Peat material will be used to dress levee slopes and could be utilized in Miami Canal tree island construction. Unsuitable material will be hauled to a certified land fill. If enough material is not available on site from the canal construction of C-624, C-624E, C-625E, C-625W, and C-626 to provide suitable levee construction material for L-624 and L-625, material may be brought from an offsite borrow area to construct the remainder of the levees. If excess material is generated from the construction, dispose of excess material first for recreation related features that require fill. Then dispose of remaining excess material by balance access roads, canal embankment improvements, or adding additional width for levee adjacent to excavation. Another disposal option would be to store the remaining excess material for future improvements onsite or either export to Miami Canal area for backfill.

A.5.1.5 Utility Relocations

Florida Power and Light lines will have to be relocated or abandoned from the center of the FEB. Additional utility lines will need to be provided for structures S-623, S-624, S-625, S-626, and S-628. The length and type will be determined during design.

A.5.2 GEOTECHNICAL DESIGN

The features presented herein represent elements of the Recommended Plan. Due to an absence of geotechnical investigation data, most of the current geotechnical design is based on assumptions about the subsurface conditions that will be encountered during construction. Preliminary or tentative design parameters and conditions presented below will need to be validated with site specific subsurface exploration which will be one of the first orders of work during the design phase. Enough construction has occurred over the years in South Florida and although the individual site conditions and the geotechnical design parameters associated with them may vary substantially, construction methods and practices used represent the general configurations that have performed satisfactorily in this region. Due to a general lack of subsurface information except at extremities of the features, detailed analyses are only of limited value for design at this stage of the project. It should also be noted that the final locations and the shape of many of the features is still in the developmental stage.

FEB structures S-625 and S-624 (culverts and siphon)

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project were assumed based on typical values for similar materials on COE projects in South Florida, empirical relationships from literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater investigations have been performed in the vicinity of this feature to date, nor are any anticipated during future design.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. It is anticipated that the culvert structure foundations will be founded on underlying limestone or compacted cohesionless fill. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. A heave or uplift evaluation will be required to design the tremie concrete slabs during the design phase. Slope stability analyses other than temporary cut slope evaluation during the design phase will not be required.

g. Excavatability analysis. Rock rippability has been estimated at this time and these estimates will be evaluated further based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the culvert and siphon structure areas can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank and through the existing embankments. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Backfill will be accomplished with compacted layers of granular backfill with rewatering. Excess excavated inorganic cobbles and grains less than 6 inches in effective diameter can be loaded onto dump trucks and hauled to the Miami Canal for canal filling. Larger cobbles and boulders can be crushed and mixed with the minus 6 inches of soil and rock.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as backfill for the structures. Excess inorganic material will be used at the Miami Canal filling area. The organic materials will then be disposed in an area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. Sheetpile cofferdams with tremie concrete slabs to facilitate dewatering and dry construction are typically incorporated into the construction of these features. Discharge of dewatering effluent will be to the canal after appropriate treatment. Dewatering is typically accomplished by sump pumps within the excavation pit with supplemental groundwater lowering via well point rows. Other methods for dewatering may be utilized for construction efficiency and cost savings.

FEB S-626 seepage pump station

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater investigations have been performed in the vicinity of this feature to date.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. It is anticipated that the pump structure foundations will be founded on underlying limestone. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after receipt of geotechnical exploration data during the design phase. A heave or uplift evaluation will be required to design the tremie concrete slabs during the design phase. Slope stability analyses other than temporary cut slope evaluation during the design phase will not be required.

g. Excavatability analysis. Rock rippability has been estimated at this time and these estimates will be evaluated further based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the culvert and siphon structure areas can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank and through the existing embankments. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Backfill will be accomplished with compacted layers of granular backfill with rewatering. Excess excavated inorganic cobbles and grains less than 6 inches in effective diameter can be loaded onto dump trucks and hauled to the Miami Canal for canal filling. Larger cobbles and boulders can be crushed and mixed with the minus 6 inches of soil and rock.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as backfill for the structures. Excess inorganic material will be delivered to and used at the Miami Canal filling area. The organic materials will be disposed in an area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. Sheetpile cofferdams with tremie concrete slabs to facilitate dewatering and dry construction are typically incorporated into the construction of these features. Discharge of dewatering effluent will be to the canal after appropriate treatment. Dewatering is typically accomplished by sump pumps within the excavation pit with supplemental groundwater lowering via well point rows. Other methods for dewatering may be utilized for construction efficiency and cost savings.

<u>New FEB Dikes and Canals</u>

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date. Geophysical testing may be utilized in the design phase to locate solution cavity locations in the perimeter dike, interior levees and the FEB bottom for treatment.

c. Groundwater Studies. Black and Veatch 2006 performed hydraulic interval testing on five boreholes in 2005. However, additional field and laboratory hydraulic conductivity tests will be performed along with 2D/3D seepage analyses of the embankment and foundation system. A pump test in the FEB may be required.

d. Recommended Instrumentation. Vibrating wire piezometers through the crest and at the toe of the exterior dike at various stations along the dike perimeter may be required as part of the design of this feature.

e. Earthquake Studies. Due to the extremely low seismicity of South Florida, adverse effects to the FEB embankments are not anticipated. Nevertheless, a liquefaction screening analysis will be performed during the design phase to verify this assumption for this feature.

f. Preliminary foundation design and slope stability analyses. The dikes will be founded on underlying limestone after stripping and removal of surficial organic peats and organic silts. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. Slope stability analyses during the design phase will be required for the steady state and end-of-construction cases. A preliminary slope stability analysis has been performed for the FEB dike embankment with results provided in Appendix A, Annex G.

g. Excavatability analysis. Rock rippability has been estimated at this time and these estimates will be evaluated further based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the dike foundation and canals areas can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Excavated inorganic cobbles and grains less than 3 inches in effective diameter can be processed and used as embankment fill. Larger cobbles and boulders can be crushed and mixed with the minus 3 inches of soil and rock. Excavation of the top organic layers in the foundation is anticipated to be in a wet condition. As the fill materials are primarily cohesionless, a single vibratory steel drum with pneumatic tires at least 5 tons is expected to be used on this site.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as fill for the inner and exterior dikes. Pervious fill (<5% fine grained material) will be used as the bridging lift between the underlying foundation limestone and the groundwater surface. The bridging lift will be placed loosely into the excavated pit and pushed forward with dozers until satisfactory bearing is achieved for dry placement. The pervious material can either be imported or derived from processing canal borrow material. Above the groundwater level, satisfactory fill from the canal borrow material shall be placed in compacted lifts in the dry. Oversize particles greater than 3 inches in nominal diameter shall be crushed into satisfactory fill or used for erosion protection features. Excess inorganic material will be destined to be delivered to the Miami Canal filling area. The organics will be disposed in an area to be determined during the design phase. These may be used later for mixing and seeding of embankment erosion protection. Riprap and bedding materials required for erosion protection will be obtained from offsite sources unless suitable rock is found on-site. Riprap and bedding will be sized during the design phase based on design water velocities. Access roads and the FEB crest road will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. From previous studies in the EAA Reservoir to the east of the FEB, and from previous core boring logs from L-24 to the west, solution cavities are expected to be encountered in foundation grade for the perimeter dike, interior levees and FEB interior. Hydraulic tests are planned to minimize seepage losses in the FEB and protect against piping losses of embankment material from beneath the dike foundation. A preliminary 2D seepage analysis and site characterization is contained in Appendix A, Annex G. A seepage analysis evaluation will be performed during the design phase which will be integrated into the slope stability analysis. Seepage control systems will be utilized depending on the results of the seepage analysis. These could include toe drains, chimney drains or cutoff walls. A piping evaluation analysis will be conducted following the seepage analysis. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. Geophysical mapping may also be used to identify the location of large voids in the dike foundation limestone and within the FEB. In the presence of such highly permeable cavities, some sort of filtering or impervious filling may be required to meet storage requirements for the FEB. Full scale pump tests in the FEB are recommended to capture the global permeability of the FEB subsurface materials. A granular filter or geotextile may be required to fill large voids lying within the dike foundation footprint to circumvent piping of embankment material through the foundation rock conduits.

A.5.2.1 General Status of Completed and Non-Executed Efforts

From experience with the adjacent EAA Reservoir A-1 facility design, there are unanswered questions as to the ability of the new FEB to retain the design storage levels. The geotechnical exploration program for the FEB will be extensive, costly and these costs should be anticipated in the budget estimates and design and construction schedules. Since no reservoir/FEB holding capacity evaluation was performed on this feature during the planning stage, it may be discovered during design that a seepage control system may be required and this eventuality should also be accounted for in the budget projections.

Summary of additional geotechnical exploration for all CEPP culverts, siphons, spillways and divide structures. Extremely limited or minimal geotechnical data is available for the site specific location of the structures in this project. Most if not all of these structures will be constructed, in part, below the groundwater table. Therefore, geotechnical explorations need to encompass dewatering features in addition to data required for facilities constructed at ground level. Each structure shall receive a minimum of two core borings consisting of standard split spoon sampling and auger drilling in soil and rock core barrel drilling in rock. At least one boring shall be deep enough to identify suitable bearing layers for deep foundations and establish the hydrogeologic properties of underlying strata for modeling purposes. Undisturbed Shelby tube samples shall be obtained for cohesive materials encountered during drilling. Laboratory index tests for soil and rock will be performed on samples obtained from the drilling. Waxed rock core samples shall be obtained to determine the rock strength and density parameters. Companion borings will be drilled alongside the core borings to conduct field hydraulic tests of the underlying strata. Laboratory tests shall also be performed on remolded samples to determine the vertical permeability of soils. In place field hydraulic tests will include specific capacity, constant head recharge and possibly slug tests.

Summary of additional geotechnical exploration for all CEPP pump station structures. Extremely limited or minimal geotechnical data is available for the site specific location of the structures in this project. Even though the existing pump station may have geotechnical data available, much of the available subsurface data was obtained using non-standard techniques and it is unlikely that the expansion of pump capacity will involve connection to the existing structure. All of these structures will be constructed, in part, below the groundwater table. Therefore, geotechnical explorations need to encompass dewatering features in addition to data required for facilities constructed at ground level. Each structure shall receive a minimum of two core borings consisting of standard split spoon sampling and auger drilling in soil and rock core barrel drilling in rock. At least one boring shall be deep enough to identify suitable bearing layers for deep foundations and establish the hydrogeologic properties of underlying strata for modeling purposes. Undisturbed Shelby tube samples shall be obtained for cohesive materials encountered during drilling. Laboratory index tests for soil and rock will be performed on samples obtained from the drilling. Waxed rock core samples shall be obtained to determine the rock strength parameters. Companion borings will be drilled alongside the core borings to conduct field hydraulic tests of the underlying strata. Laboratory tests shall also be performed on remolded samples to determine the vertical permeability of soils. In place field hydraulic tests will include specific capacity, constant head recharge and possibly slug tests.

Summary of additional geotechnical exploration for the FEB embankments and all CEPP canals. Extremely limited or minimal geotechnical data is available for the site specific location of the FEB dike and interior levees and canals in this project. Much of the available subsurface data was obtained using non-standard techniques and is located outside the footprint of the FEB and dike/canal system. It is foreseen that the surficial organic materials under the dike footprint will be removed and replaced with a pervious fill bridging lift below the groundwater table. Therefore, geotechnical explorations need to encompass dewatering features in addition to data required for facilities constructed at ground level. The FEB embankments will receive a minimum core boring at the crest and toe at a tentative spacing of 1000 feet. These borings shall consist of standard split spoon sampling and auger drilling in soil and rock core barrel drilling in rock. Borings shall be deep enough to identify soft layers in the foundation and establish the hydrogeologic properties of underlying strata for modeling purposes. Undisturbed Shelby tube samples shall be obtained for cohesive materials encountered during drilling. Laboratory index tests for soil and rock will be performed on samples obtained from the drilling. Waxed rock core samples shall be obtained to determine the rock strength parameters. Companion borings will be drilled alongside the core borings to conduct field hydraulic tests of the underlying strata. Laboratory tests shall also be performed on remolded samples to determine the vertical permeability of soils. In place field hydraulic tests will include specific capacity, constant head recharge and possibly slug tests. A full scale pumping test with monitoring wells is recommended to be performed to establish the overall transmissivity of the FEB subsurface. Areas of canal excavation widening and deepening shall be explored with test pits. Samples shall be taken of excavated material for index testing as excavated material will become fill for other features of this project. Rip ability tests will be conducted in conjunction with the test pit exploration. Sporadic shallow core borings shall be drilled in the canal area to the depth of the canal to obtain rock samples for unconfined compression testing. Geophysical testing may be used to identify and map solution cavities in the FEB footprint and within the dike alignment. A test fill for embankment construction feasibility in the wet and a breakdown analysis of excavated limestone and soil after compaction will also be evaluated during the exploration program. Rock core samples may be mapped by digital photography to stochastically estimate porosity, permeability and filtration characteristics of limestone layers.

A.5.2.2 Soils

The soils in the EAA are primarily composed of peats and mucks (Bottcher 1994). Deep, clean sands characterize the area east of the Everglades and to the South of Lake Okeechobee with wet, gray or grayish-brown, sandy soils underlain by sandy clay cover the area west of the Everglades. The peat and muck soils, which are dark brown to nearly black, cover approximately 90 percent of the area being considered in the study area. They were formed in marshes or swamps by the partial decay of plant materials, with some admixture of mineral soil in the case of muck. Peat generally, consists of 65 percent or more organic material with relatively little mineral matter. Muck on the other hand, consists of 25 to 65 percent plant material mixed with sand, silt, and clay. The peat and muck soils may differ from each other in the kind of plant material that they contain, in the corresponding depths, and/or in the nature of the underlying material. The peat and muck may rest directly on limestone or on an intermediate layer of sand or marl.

The highly organic soils have been divided into four types: Okeechobee muck, Okeelanta peaty muck, Everglades peaty muck, and Everglades peat.

1) Okeechobee muck is a nearly black mixture of organic material and fine mineral soil. The organic portion of the soil is formed from the remains of water plants, while the mineral content probably results from the deposition of fine sediment during overflows from Lake Okeechobee.
2) Okeelanta peaty muck consists of finely fibrous, well-decomposed organic matter over a layer of black plastic muck; it usually overlies hard limestone.

3) Everglades peaty muck contains somewhat less mineral matter than Okeelanta peaty muck. The surface layer rests on brown, fibrous peat, and it usually lacks the subsurface layer of black plastic muck.
4) Everglades peat, the most extensive of the organic soils, is formed mostly from partially decayed sawgrass. The upper 12 inches is a nearly black, finely fibrous peat which contains approximately 10 percent mineral soil. The subsoil is brown, fibrous peat which rests on the underlying rock, sand, or marl.

Most of the characteristics, properties, and composition of the muck and peat soils depend on the fact that those types of soils are essentially mixtures of water and partly decomposed plant materials. When saturated, the soil is a little heavier than water. One of the outstanding characteristics of the peat soil is its light weight when dry. The oven-dry weight of peat is about 7 pounds per cubic foot, and the mineral content is about 10 to 15 percent by weight of the dry material. Another important property is the high shrinkage value. Peat soils will shrink as much as 75 percent of their original volume when dried, and will not expand to their original volume when water is added.

Another important property is their high propensity for water retention. Peats vary considerably in that respect, depending on their origin, degree of decomposition, and chemical composition. While a dry mineral soil will absorb and hold from 20% to 40% its weight of water, a peat soil will retain many times its dry weight of moisture, depending on conditions. On an oven-dry weight basis, some of the peats have as much as 1,200 percent water when saturated, with the average having about 750 percent.

Laboratory permeability tests and field pumping tests indicate that seepage through peat soil is much greater vertically than horizontally. That can reasonably be attributed to the fibrous nature of the soil and its characteristic vertical root channels. Peat and muck material presented in less recent geotechnical exploration reports provide a general idea of the thickness of organic surface materials in the region. However, there are selected areas where the organic soil has been reduced due to recent construction, development, fire, erosion, compression, or removal. In other areas, there may be accretion of organic materials. Where peat is encountered in the borrow area, it would be removed and not used as construction material. The available geotechnical information indicate suitable materials for embankment construction and other fills, mainly interbedded sands and/or marls with limestone, are available throughout the project area. In some areas, in-situ materials may have to be processed to achieve feature performance requirements.

Seepage movement in the Everglades is largely through the porous rock and sands beneath the peat. . In some areas, marl, consisting of fine sandy silt derived from neighboring limestone occurs. Marl soils tend to be quite impermeable, and act as a seal that retards movement of water.

The sands, in general, are fine-grained and poorly graded having intermediate coefficients of permeability. The marl soils are widely distributed under the organic soils, and in places are consolidated into a hard limestone just under the peat. Usually, however, the marl is a soft, grayish-white, calcareous silt of fresh-water origin. Other marls, with inclusions of sand, silt, clay, and shell, appear within the area.

A.5.2.3 Geology

The character of the marginal marine sediments changes from north to south of the Redline. North of the Red Line within Palm Beach County, the sediment thickness consists of poorly consolidated marine limestone, quartz sandstone, and sandy limestone with abundant mollusk fossils (Reese and Wacker, 2009) and is known as the Fort Thompson Formation.

A surficial aquifer system in the southeastern portion of the Redline is wedge shaped, thickening eastward toward the Atlantic Ocean. The surficial aquifer system (Fish, 1988; Fish and Stewart, 1991) comprises a sequence of highly permeable limestone, quartz sand, shell, and terrigeneous mudstone of Pliocene to Holocene age **Figure A-1**). The sand content of the surficial aquifer system is high in the far southeastern corner of the EAA.

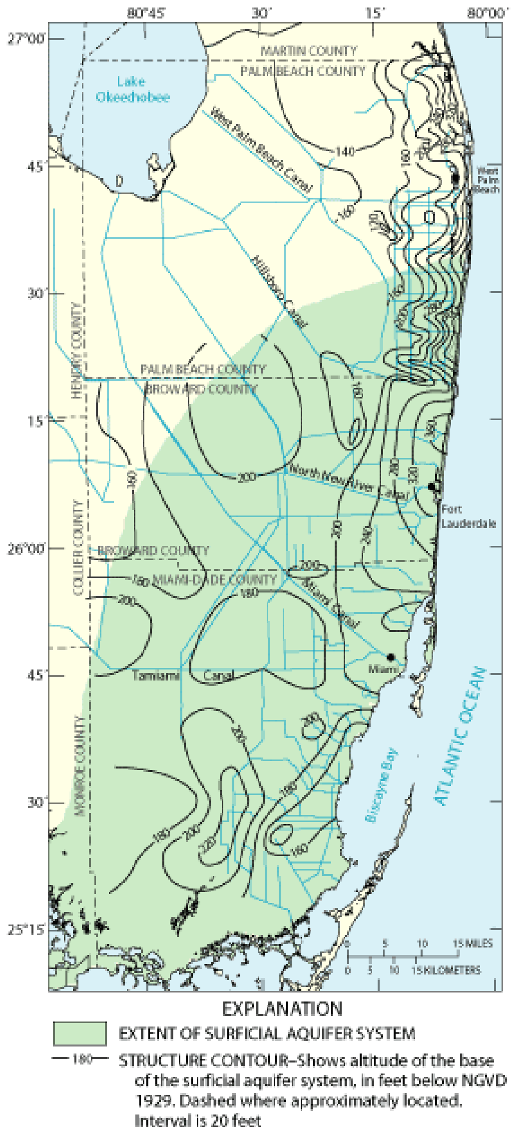


FIGURE A-1. SURFICIAL AQUIFER SYSTEM

Base and extent of the surficial aquifer system in south eastern Florida. Modified from Fish (1988), Shine and others (1989), and Fish and Stewart (1991).

The surficial aquifer system has been divided into separate aquifers and semiconfining (leaky) units of quartz sand, terrigeneous mudstone, and limestone (Fish, 1988; Fish and Stewart, 1991). The Fort Thompson Formation, Anastasia Formation, and Key Largo Limestone yield the most water and constitute the prolific Biscayne aquifer. The Biscayne aquifer does not extend into central and northern Palm Beach County but does encompass the very southeastern portion of the EAA north of the Red Line.

Although karst features have not been documented in Palm Beach County, a “cavity-riddled” zone (Fischer, 1980) is likely associated with karst dissolution.

Prior to the development of the EAA, peat deposits (resting on top of the surficial aquifer) ranged from 6 to 17 ft thick (Stephens and Johnson, 1951). Peat deposits thinned where they extend southward south of the Redline. Near the boundary of the Red Line (the northernmost part of the Everglades), a calcareous marl sequence separates the peat deposit from underlying limestone bedrock; freshwater marl is considered indicative of a shorter hydroperiod that cannot support peat accumulation (Gleason and others, 1984). Drainage works in agricultural areas designed to control flooding south of Lake Okeechobee (north of the Redline) have contributed greatly to oxidation- and compaction-driven subsidence of peat deposits (Renken and others, 2013). Subsidence ranged from 3 to 9 ft in the EAA.

Black and Veatch, 2006 conducted a massive three-volume study for South Florida Water Management District by performing nearly 235 borings and geotechnical laboratory tests within the A-1 FEB to characterize subsurface conditions for embankment design, embankment stability, settlement, seepage analyses and to provide information for identifying potential borrow materials (ANNEX G-1). Some of the borings were converted to piezometers. Generalized results of the geotechnical investigation revealed the following:

*Limestone caprock that was interpreted to be top of Ft. Thompson Formation. Most thicknesses ranged from 3.5 to 6 feet although thickness to 9.2 feet is not uncommon. Unconfined compressive strengths ranged from 433 to 9,768 pounds per square inch (psi) with an average of 2,938 psi.

*Silty sand (below caprock) had averaged carbonate content of 83.6 percent.

*Lower limestone layer (below silty sand) varies 1 to 6 feet thick. Average Rock Quality Designation (RQD) was 18.5 percent.

*Shelly, fine, uniform, subrounded quartz sand (below lower limestone layer) interpreted to part of the Caloosahatchee Formation and Pinecrest Sand.

*Interpreted Ochopee Limestone (below shelly quartz sand). Average depth to top of limestone was 74 feet.

*Unnamed Sand Formation (below Ochopee Limestone) consisting of silty sand.

Because the volume of borings and laboratory testing is so great, the reader is referred to the Black and Veatch 2006 three-volume report for more details.

Ardaman and Associates, 2002 conducted one known boring within the proposed A-2 FEB footprint in the EAA to 180 feet depth (ANNEX G-1). The shallow portion of the boring indicates a massive hard limestone from 8.8 ft to -12.0 ft North American Vertical Datum of 1988 (NAVD88). Below this rock unit are interbeds of sand (poorly graded and silty) sitting on top of a thin limestone layer. From -90 ft to -131 ft NAVD is a massive sandstone unit which overlies a poorly graded sand layer.

Despite the geological information presented north of the Red Line, significant data gaps do exist. Further geological investigations are needed for the proposed A-2 FEB for the proposed structures for this area.

A.5.2.4 HTRW

The A-2 FEB is a 14,000 acres parcel of land. The land is presently dry and it is proposed to be inundated with water.

South Florida Water Management District (SFWMD) completed a draft Summary Environmental Report for the A-2 FEB, dated 21 August 2012. The Summary Environmental Report documents that all known point sources on the property have been addressed. The Florida Department of Environmental Protection (FDEP) has issued Site Rehabilitation Completion Orders (SRCO) for all known point sources within the project boundary. A copy of this report is included in the main PIR, **Annex H**.

To address the lack of sampling results for the cultivated areas of the A-2 parcel, the SFWMD conducted limited soil sampling in the winter/spring of 2013. With agreement of the USFWS, the sampling density was set at 10 percent of the 50 acre grids rather than the typical 30 to 50 percent typically specified per the *Protocol for Assessment, Remediation, and Post-Remediation Monitoring for Environmental Contamination on Everglades Restoration Projects* (the ERA Protocol), dated March 13, 2008 (A copy of this protocol is in the main PIR, **Annex H**. SFWMD analyzed 30 composite samples from the 14,000 acre site for pesticides, herbicides, total organic carbon and metals following a stratified random approach. The laboratory result indicate that some of the site soils have residual arsenic, barium, cadmium, chromium, copper, mercury, selenium, 2-4-D, atrazine, metribuzin, phorate, and dieldrin. The USFWS and FDEP have preliminarily determined that the residual agricultural chemicals found on the A-2 FEB lands do not present a risk to protected resources. Based on the results of the 2013 soil testing, the USFWS and FDEP are recommending that during the initial operations of the FEB, the SFWMD perform testing of water for several contaminants (2,4, D, atrazine, metribuzin, phorate, dieldrin, chromium, mercury, selenium, copper) as well as testing of periphyton and apple snails for copper. The FDEP also recommended the development of a soil management plan to address the fate of arsenic impacted soils during construction as well as the same start-up operations sampling program as provided by the USFWS. The FDEP and the USFWS both recommended that agrochemical best management practices be instituted during the continued cultivation of the lands.

The A-2 lands will remain in agricultural production for several years until the A-2 FEB is set for construction at which time the agricultural leases will be terminated. Once farming has ceased on the project lands, an Exit Assessment will be performed to determine the presence of any new potential sources of HTRW since the completion of the previous Phase II ESA, and to verify the concentration of contaminants in the cultivated areas at selected locations. The results of these audits will be provided to the FDEP and USFWS for their review, comment, and concurrence regarding the need for remedial actions. The assessment of the project in relation to the CERP Residual Agricultural policy is included in the main PIR **Appendix C.2.2**. Should remediation of HTRW contamination be required, it is the responsibility of the SFWMD, the non-Federal, sponsor and is not a creditable cost to the project.

A.5.3 HYDRAULIC DESIGN

A.5.3.1 General Status of Completed and Non-Executed Efforts

Features identified in the Recommended Plan have been designed to the level of detail necessary to provide cost estimates and determine feasibility of hydraulic design. All components north of the redline have been identified and sized appropriately according to available modeling data, historic information, and best engineering judgment. All project components will be optimized during PED phase for cost efficiency and performance, incorporating updated data and information as it becomes available. General hydraulic design of all identified components north of the redline are described in the following sections.

A.5.3.2 Hydraulic Design - General

This section provides a brief overview of the hydraulic design criteria, parameters, and intent/purpose of project features. Detailed hydraulic design of individual components is described in later sections, including hydraulic design data sheets. Detailed analysis resulting from model simulations may be found separately in Appendix A, Annex A-1. Currently, all elevations are referenced to NGVD 29; elevations will be provided in both NGVD 29 and NAVD 88 when revised during PED.

The Value Engineering (VE) workshop held in February 2013 generated some new ideas, some with less cost, and some with greater cost. However, workshop results and conclusions were not addressed nor included in the hydraulic design appendix herein due to time and schedule constraints. Refer to Section A.10 Value Engineering for more information.

A.5.3.2.1 Design Criteria and Parameters

A.5.3.2.1.1 Canals

Canal side slopes are generally steeper than is found in many sandy regions. This is due to the limestone geology allowing for near vertical slopes in some locations. Generally, there is a preference for some slope in case of sand lenses and well weathered limestone that with time deteriorates to gravel and sand sizes, therefore most canal side slopes are 1V:2H.

A.5.3.2.1.1.1 Manning's Roughness Coefficient Determination

A Manning's roughness coefficient, or n value, of 0.035 was used for canal design in conveyance of design flows. The Palm Beach County region where the CEPP components and canals will be constructed has geological features characteristically described as limestone covered with a relatively thin layer of overburden (peat, marl, muck). The canals will most likely be constructed by blasting with excavation or possible dragline operations. These types of excavation methods leave a relatively coarse bottom and bank with sharp edged rocks and rubble. With canal age or maturation, some of these irregularities will become less defined resulting in a smoother perimeter with lower roughness values. Aquatic growth is expected along the upper banks where the overburden lies, but the design depths should be relatively free of plant roots extending from bottom to surface obstructing flow. Floating plants will be controlled by spray and harvest methods. The Manning's n value used was obtained from investigating various sources and noting them as follows.

In C&SF Project General Studies and Reports, Part I, Supplement 18, the following was noted; a value of at least 0.035 should be used where channels are constructed primarily in rock. This value is for channels with no appreciable erosion and with rapid-growth vegetation along the upper banks because of the organic soil overburden. Other sources provide Manning's n values within the same ranges as SCS and USGS for similar type canals. Brater and King's Handbook of Hydraulics, 7th ed., provides an n value of 0.035 for canals with rough stony beds and weeds on earth banks in fair condition. From the preceding investigations, and experience at the Jacksonville District in Florida, a Manning's n value of 0.035 appears to be appropriately applied to the design to satisfy criteria outlined by all referenced sources as the minimum acceptable value.

In detail design phase, the canal parameters may be modified for optimal benefit cost ratio.

A.5.3.2.1.2 Head Loss

Due to the relatively flat topography throughout the project area, the hydraulic head losses across many of the control structures are low, resulting in the design of larger structures (number and size of barrels, bays, etc.) than may typically be assumed for other regions. The use of pumps was avoided wherever possible to reduce operation and perpetual maintenance costs. During PED phase, USACE Jacksonville District (SAJ) expects to optimize system operations and therefore structure sizes for cost and performance efficiencies.

A.5.3.2.1.3 Flow and Velocity

Design flow rates for all water control structures were determined based on Regional Simulation Model for Basins (RSM-BN) model outputs and existing canal and structure capacities. To capture cost impact adequately, structures and canals were designed for maximum capacity scenarios. Optimization of these features will be conducted during the PED phase for performance and cost efficiency.

Canals were designed to maintain a velocity of 2.0 fps or less to avoid potential erosion damage. Given the small topographical relief of the project area, this is typically the condition under normal operations, regardless.

A.5.3.2.1.4 Water Control Structures

The proposed plan north of the redline will have multiple water control structures throughout the project areal extent (S-623, and FEB structures S-624, S-625, S-626, S-627, S-628). The function of the control structures are for impoundment inlet, impoundment outlet discharge to major canals, discharge into the adjacent A-1 FEB, and seepage return. The A-2 FEB inlet structure, S-624, was sized to match the existing in-bank capacity of the Miami Canal of 1,550 cfs when diverting flows from Lake Okeechobee into the FEB. This value was determined by using Miami Canal survey data previously collected for prior EAA studies. That capacity for the discharge structure S-625 was also determined based on this data, as well as RSM-BN model results. The inflow/outflow structure S-628, which hydraulically connects A-1 and A-2 FEBs was designed for a total capacity of 930 cfs. Structure S-623 is a gated spillway in line with the STA 3/4 Supply Canal, at the intersection with the Miami Canal. The structure will serve as a divide structure when the A-2 FEB discharges into the existing STA 3/4 Supply Canal. During these discharge periods, S-623 will be closed to prevent Miami Canal flows from mixing with pretreated FEB discharges for water quality objectives. S-623 has been sized to match the capacity of existing G-372 at 3,700 cfs. Seepage collection will be obtained using seepage pump S-626, which is sized to 700 cfs based on estimated seepage rates and design redundancy.

Hydraulic design data sheets for all structures are located at the end of the A.5.3 Hydraulic Design section. Detailed analysis of these structures can be found in Appendix A, Annex A-1.

A.5.3.2.1.4.1 Gated Culverts

The entire CEPP project includes numerous gated box culverts across the entire project area. Construction material for all culverts is to be cast in place concrete.

An entrance loss coefficient value of 0.9 (assumed due to gate-added turbulence around inlet) and exit loss coefficient of 1.0 was used for all gated culvert structures. Also, the Manning’s friction or energy loss coefficient was assigned 0.013 for all culverts. All major conveyance culverts were designed to remain submerged year round to reduce aquatic growth within, thereby better maintaining design friction head losses. All gated culvert sites were designed with a minimum of two culverts to allow maintenance activities to coincide, however with reduced capacity operations.

No attempt has been made to date in standardizing gates sizes to the extent possible due to report time and schedule constraints. For example, one box culvert structure calls for 3-9x9 gates and another nearby calls for 2-11x11 gates. Potentially, the 3-9x9 purposes can be met with 2-11x11 gates as well. This may reduce overall number of gates, reduce number of gate-types, and required spares and training; thereby, having less long-term O&M costs. During PED, the final design will include gate/culvert size optimization as much as possible to reduce long-term costs and promote efficiency in the non-Federal sponsor maintenance program.

A.5.3.2.1.4.2 Gated Spillways

The CEPP Recommended Plan includes the design of an ogee weir concrete spillway with steel vertical lift gates located in line with STA 3/4 Supply Canal, at the intersection with the Miami Canal.. The spillway was designed to be in conformity of engineering guidance found in USACE EM-1110-2-1603. All ogee spillways have vertical gates for controlled discharge operations. The spillway was designed with minimal head differential for conveyance energy. The S-623 spillway was designed with a 0.1 foot head differential. This was a design constraint supported by the following reasons: (1) flat terrain topography, and (2) Miami Canal's control stage range limitations given the proposed location of the spillway. SAJ acknowledges this low design head differential constraint and optimization will be required, including Value Engineering appropriate structure type for this function.

A.5.3.2.1.4.3 Emergency Discharge Structures

Emergency overflow spillways are non-gated non-mechanical structures that do not require human intervention for uncontrolled discharge operations. The benefits of an emergency overflow spillway for impoundments are primarily twofold: one being economics and the other being safety for the public downstream of a potential, though highly improbable dam breach. The spillway allows excess water to be discharged from the impoundment, thus lowering the maximum surcharge pool level where the superiority is measured from the design conditions applied, i.e. wind speed(s). However, the implementation of an emergency overflow spillway removes errors in human operations and mechanical failures from the equation for known causes of some historic dam failures. Also, should some particular or sequence of events unforeseen occur because of remote possibilities, it is considered far more advantageous to allow excess water out before a catastrophic breach can occur. The S-627 overflow spillway serves as the A-2 FEB emergency discharge structure.

A.5.3.2.1.4.4 Pump Stations

The CEPP Recommended Plan proposes to construct a new seepage collection pump station, S-626, for seepage management. The pumping rate of 500 cfs was established to accommodate the peak estimated seepage inflow rate of 400 cfs, as well as provide additional capacity for possible high flow events. Additionally, the G-370 and G-372 pump stations will be used to serve as FEB inflow pump stations.

A.5.3.2.1.5 Embankments

The FEB at this time carries a low hazard potential classification (HPC) per DCM-1, which is extended to embankment design. Embankment top widths are 14 feet wide per DCM-4, with dam heights based on analysis of the following criteria (ER-1110-8-2(FR), ER-1110-2-1156, DCM-2, and risk). Section A.5.3.3.2.1.5 Other Features, Emergency Overflow Spillway discusses the details of the flood routing for design in greater depth.

1. Three feet above the maximum surcharge pool elevation. The maximum surcharge pool elevation is based on the greatest elevation resulting from the following storm routings:
    a. The Inflow Design Flood (IDF), which is identified as the 100-yr 24-hr storm event for the CEPP FEB, per DCM-2;
    b. The 50% 72-hr PMP per ER-1110-8-2(FR); and

Wind setup and wave run-up analysis on critical fetch lengths with the impoundment at full pool. Wave run-up is dependent on levee slope (steeper slopes have higher run-up).

A.5.3.3 Flow Equalization Basin

A.5.3.3.1 General Information

The CEPP features north of the redline consist of the construction of an FEB and associated infrastructure. All north of the redline components are within Palm Beach County, north of Water Conservation Area (WCA) 3A.

The CEPP project features a 14,000 ac 4-foot deep, above ground flow equalization basin. Other design features include inflow canals, collection canal, outflow canal, seepage pump stations, gated culverts, gated spillway, an emergency overflow spillway, and a perimeter seepage control canal.

TABLE A-2. A-2 FEB DESIGN ELEVATIONS

Parameter	Elevation, ft NGVD (NAVD)
Top of levee	20.30 (18.9)
Average Natural Grade	9.00 (7.6)
Maximum Surcharge Pool	15.15 (13.75)
Maximum Normal Pool	13.00 (11.6)

*Levee crest elevation of 20.30 ft NGVD was used throughout the features based on available information at the time of design. Revisions made to the levee crest elevation will be incorporated into the final design during PED.

TABLE A-3. A-2 FEB STORAGE CALCULATIONS

Storage Area	14,000 acres
Maximum Normal Pool Depth	4 feet
Storage	56,000 ac-ft
Fill/Drawdown Rate at 1,550 cfs at 4 ft depth	0.22 ft/day
Time to Fill/Drawdown 4 feet at 1,550 cfs	18.2 days

A.5.3.3.1.1 Purpose

The purpose of the A-2 FEB, which will be operated in conjunction with the use of the A-1 FEB (designed and constructed by the SFWMD) is to capture additional water from Lake Okeechobee for delivery to the Everglades, while maintaining the capability to treat the existing EAA runoff and limited Lake

Okeechobee discharges. The integrated FEB operations will be able to accept and provide some water quality pre-treatment of additional water from Lake Okeechobee during off-peak times, such as the dry season, when treatment capacity is available in the downstream STAs.

A.5.3.3.1.2 Location

The A-2 FEB is located in Palm Beach County, between the Miami Canal and North New River Canal, and north of Water Conservation Area (WCA) 3A. It is adjacent to the western boundary of the A-1 FEB. The inflow to the impoundment begins about 1.5 miles east of G-372 pump station.

A.5.3.3.1.3 Features

The CEPP project has the following features north of the redline:

Structures:
S-624 Gated Culvert (DS-5)
S-625 Gated Culvert (DS-7)
S-623 Gated Spillway (DS-8)
S-628 Gated Culvert (DS-9)

Canals:
C-624 FEB Inflow Canal
C-624E FEB Spreader Canal
C-625E FEB Collection Canal
C-625W FEB Discharge Canal
C-626 FEB Seepage Collection Canal

Pump Stations:
S-626 Seepage Collection Pump

Other Features:
S-627 Emergency Overflow Spillway (CS-4)

Figure A-2 illustrates all feature locations for North of the Redline (structures and canals are not to scale or geographically referenced). Figure A-2.1 shows the A-1 FEB Final Layout. Detailed design analysis for all hydraulic components north of the redline can be found in supplemental documents located in Appendix A, Annex A-1.

FIGURE A-2. A-2 FEB LAYOUT

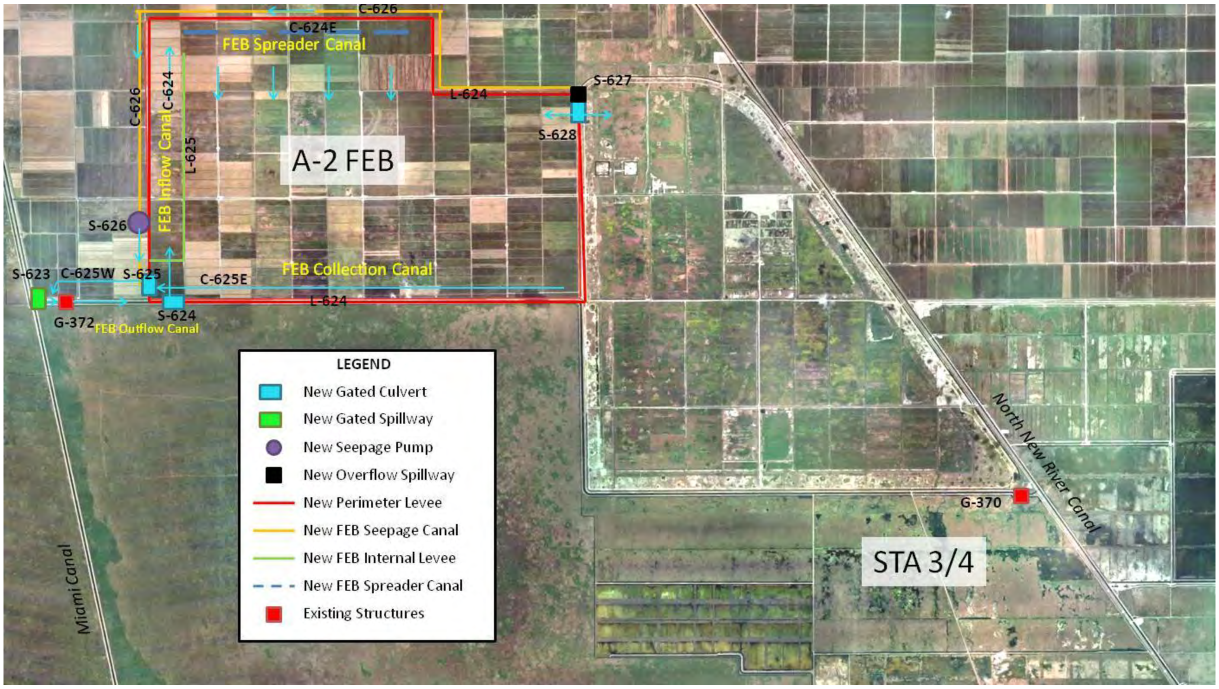
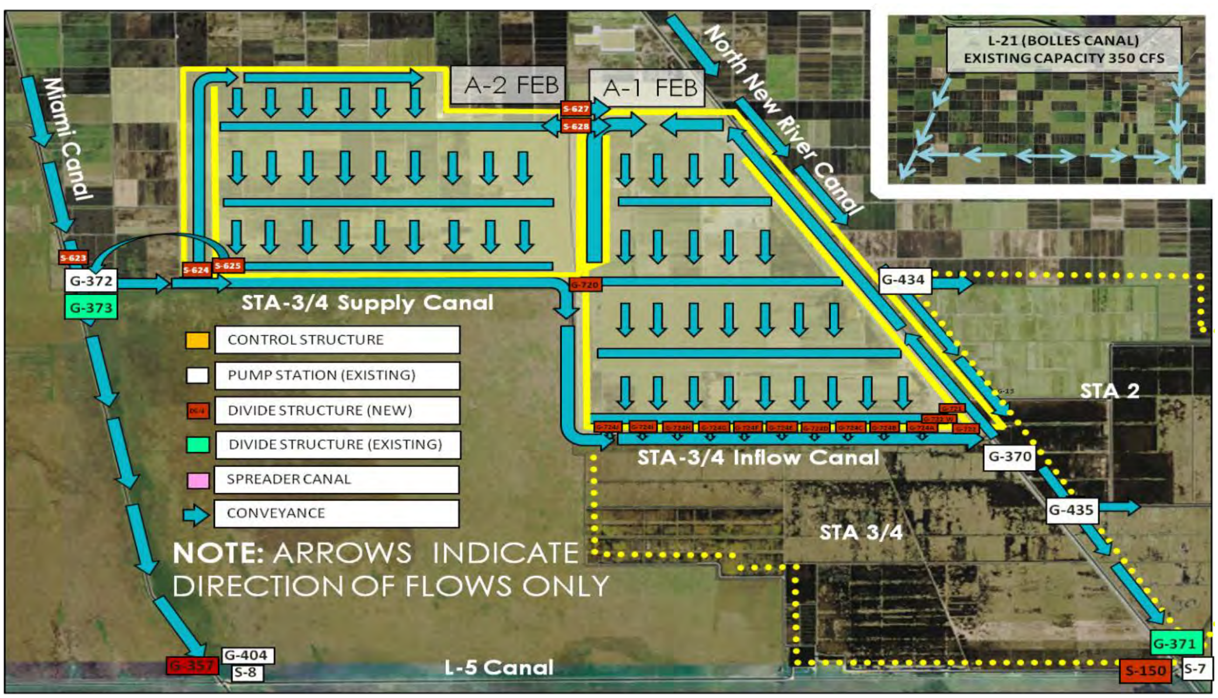


FIGURE A-2.1 FINAL A-2 FEB LAYOUT WITH EXISTING A-1 FEB



A.5.3.3.2 Hydraulic Design

A.5.3.3.2.1 Proposed Water Control Structures

A.5.3.3.2.1.1 Gated Culverts

S-624 Gated Culvert

The S-624 structure is a gated sag culvert (inverted siphon) that serves as the controlled inflow into the A-2 FEB. This structure will operate in conjunction with the existing G-372 pump station to route flows from the Miami Canal into the impoundment. The structure will open for inflow operations into the FEB from G-372, and will close during A-2 FEB by-pass operations (flow directly to STA 3/4 or the A-1 FEB) or to prevent back flow into STA 3/4 Supply Canal. S-624 is a two-barreled, gated sag culvert with four 45 degree bends. The culverts will run from the STA 3/4 Supply canal, beneath the FEB discharge/collection canal, and into the FEB inflow canal/flowway. The design flow is 1,550 cfs with a design hydraulic head of 2.5 feet resulting in a design velocity of 6.75 fps. This velocity was targeted in design to provide a scour velocity to clean out culverts, thereby reducing periodic maintenance requirements. The structure is a two barreled cast-in-place concrete box culvert with dimensions of 11 ft by 11 ft each with vertical slide gates, and a total length of approximately 350 ft. The upstream invert is set at elevation -4.50 ft NGVD, 0.5 feet above the existing bottom elevation of the STA 3/4 Supply Canal. The downstream invert is set at elevation 0.50 ft NGVD, 0.5 feet above the proposed bottom elevation of the FEB Inflow Canal. The S-624 structure is designed to cross beneath the existing collection canal (invert elev. 0.00 ft NGVD) with a vertical clearance of 3 feet, resulting in a minimum barrel sag invert of -14.00 ft NGVD. S-624 is located near the southwest corner of the A-2 FEB, east of the G-372 pump station.

S-625 Gated Culvert

S-625 is a discharge structure from the A-2 FEB. This structure will open to allow for the FEB to discharge towards the headwaters of the G-372 pump station to provide hydraulic lift for redistribution through the STA 3/4 Supply Canal and for delivery to STA 3/4. S-625 is a three barreled gated box culvert structure with dimensions of 9 ft by 9 ft with vertical slide gates, and total length of 140 feet. The upstream invert is set at elevation 0.50 ft NGVD, 0.5 feet above the invert of the existing collection canal. The downstream invert is set at elevation 0.50 ft NGVD immediately entering the FEB Outflow Canal; however the canal tapers to a bottom elevation of -5.00 ft NGVD at a slope of 1V:5H. The design flow is 1,550 cfs with a design hydraulic head of 1.0 foot. The design velocity through the structure is 6.5 fps. S-625 is located in the southwest corner of the FEB in line with the western perimeter levee.

S-628 Gated Culvert

S-628 is a bi-directional inlet and outlet structure that hydraulically connects the A-2 FEB to the A-1 FEB. This feature will allow water to be passed between the A-2 and A-1 FEBs, depending on impoundment stages and capacity. Water from the Miami Canal could potentially be routed through the A-1 FEB by use of this structure. The opposite operation can occur, using water routed through A-1 from the North New River Canal via G-370 pump station and G-15 to supplement water in A-2. S-628 is a two-barreled gated box culvert with dimensions of 9 ft by 9 ft with vertical slide gates. The design flow is 930 cfs (60% of total A-2 inflow, assuming only partial flow would be conveyed between impoundments) with a design hydraulic head of 1.0 foot. The upstream and downstream barrel inverts are set at elevation 1.50 ft NGVD. The design velocity through the structure is 5.75 fps. S-628 is located in the northeast corner of the A-2 FEB, in line with the A-2 eastern perimeter levee. During PED, the feasibility of utilizing a single 11 ft by 11 ft box or vertical sluice gates (spillway) will be investigated.

A.5.3.3.2.1.2 Gated Spillways

<u>S-623 Gated Spillway</u>

The S-623 spillway will serve as a divide structure to separate pre-treated FEB waters from untreated waters of the Miami Canal to maximize incidental water quality value of the flow-through impoundment. When open, S-623 will allow for the normal operations of the G-372 pump station to route Miami Canal water, or when closed can be used to route pre-treated FEB water through the STA 3/4 Supply Canal to STA 3/4. S-623 is a four-bay gated spillway. The design flow is 3,700 cfs with a design hydraulic head of 0.25 feet. The design flow was established to match the existing capacity of the G-372 pump station downstream at 3,700 cfs. The spillway consists of four gates with dimensions of 35 ft wide by 14 ft high. The crest invert elevation is set to 3.50 ft NGVD. The upstream and downstream aprons are set at an elevation of -2.00 ft NGVD, with an apron length of 36 feet. S-623 is located in line with the STA 3/4 Supply Canal, west of the G-372 pump station. During PED, a Value Engineering investigation will be performed to optimize structure type and size for this design function.

A.5.3.3.2.1.3 Canals

<u>C-624 FEB Inflow Canal</u>

The C-624 canal is the A-2 FEB inflow canal. The canal is located east of the G-372 pump station inside the A-2 FEB western boundary. The canal is excavated in the flowway between the exterior FEB perimeter levee and the interior levee. The canal has a bottom width of 40 feet, with 1V:2H slopes up to the top of bank at natural grade (9.00 ft NGVD), with a 20 foot wide bench on both sides. The levees extend from the bench at a slope of 1V:3H. The Manning's n value for banks and channel was 0.05 and 0.035, respectively. Design data for C-624 is summarized in **Table A-4** and **Table A-5**. The length of the canal is approximately 4 miles (21,120 feet), beginning at the FEB interior levee and ending at the northern boundary of the FEB, which transitions into C-624E Spreader Canal.

TABLE A-4. C-624 GRAVITY INFLOW CANAL

Design HW	Design TW	Length	Side Slope L/R	Canal Bottom Width	Natural Ground	Canal Bottom Elevation	Average Depth
Ft, NGVD	Ft, NGVD	Feet	1V:?H	Feet	Ft, NGVD	Ft, NGVD	Feet
14.25	13.00	21,120	1:2/1:2	40	9.00	0.00	9.00

TABLE A-5. C-624 GRAVITY INFLOW CANAL
SUMMARY OF HYDRAULIC DESIGN DATA

Station	Design Water Surface Elevation	Flow Area	Mean Channel Velocity
	Ft, NGVD	sq ft	fps
RS 0 (Downstream)	13.00	1034.00	1.74
RS 4224	13.30	1076.65	1.68
RS 8448	13.58	1115.70	1.63
RS 12672	13.83	1151.84	1.58
RS 16896	14.06	1185.56	1.55
RS 21120 (Upstream)	14.27	1217.21	1.51

C-624E FEB Spreader Canal

The C-624E canal is a spreader canal that runs along the northern boundary of the A-2 FEB for approximately 4 miles. The canal will receive flows from the FEB inflow canal and will create an even distribution of water cross the northern part of the impoundment. No head losses were assumed for this canal; the water surface profile was designed to be uniform throughout the length of the canal in order to achieve the uniform distribution. The right bank (southern bank) of the spreader canal was set at elevation 9.25 ft NGVD to provide a small berm above natural grade (9.0 ft NGVD), creating a small driving head to create sheet flow downstream at shallow depths. Design data for C-624E is summarized in **Table A-6**. Optimization of the spreader canal design will be conducted during PED.

TABLE A-6. C-624E SPREADER CANAL

Length	Side Slope L/R (N/S)	Canal Bottom Width	Average Ground	Canal Bottom Elevation	Left bank (north) elevation	Right bank (south) elevation
Feet	1V on ?H	Feet	Ft, NGVD	Ft, NGVD	Ft, NGVD	Ft, NGVD
21,120	1:2/1:2	275	9.00	-10.00	9.00	9.25

C-625E FEB Collection/Discharge Canal

The C-625E canal serves as the FEB collection canal along the southern boundary of the FEB. The canal is currently the seepage canal for the existing STA 3/4 Supply Canal. When stages in the FEB are low, sheet flow will collect in C-625E and will be conveyed to S-625 discharge structure. When the FEB experiences greater depths, the C-625E will be completely submerged, but will still provide conveyance assistance to the S-625. Existing data for C-625E is summarized in **Table A-7**. The C-625E canal template will not be modified within the FEB footprint.

TABLE A-7. C-625E COLLECTION CANAL EXISTING CONDITIONS

Length	Side Slopes	Canal Bottom Width	Canal Bottom Elevation	Left bank (south) elevation	Right bank (north) elevation
Miles	1V on ?H	Feet	Ft, NGVD	Ft, NGVD	Ft, NGVD
6.0	1:2	10	0.00	20.30	9.00

C-625W FEB Discharge Canal

The C-625W canal serves as the FEB discharge canal, extending from the S-625 discharge structure to the headwater of G-372 pump station. The existing canal currently serves as the seepage canal for the STA 3/4 Supply Canal, but will be modified to accommodate the FEB discharges. The existing canal will be extended northward and westward of the G-372 pump station to create a tie-in at the headwater of the structure. The canal will have a 1V:5H transition from elevation 0.0 ft NGVD where outlet structure S-625 ties into the canal, down to elevation -5.0 ft NGVD for conveyance capacity purposes. Design data for C-625W is summarized in **Table A-8** and **Table A-9**.

TABLE A-8. C-625W FEB DISCHARGE CANAL

Design HW	Design TW	Length	Side Slopes	Canal Bottom Width	Natural Ground	Canal Bottom Elevation	Average Depth
Ft, NGVD	Ft, NGVD	Feet	1V:?H	Feet	Ft, NGVD	Ft, NGVD	Feet
11.00	10.0	7,900	1:2	20	9.00	-5.00	14.0

TABLE A-9. C-625W FEB DISCHARGE CANAL COMPARISON OF CANAL IMPROVEMENTS

Station	Existing Canal at 1550 cfs				Improved Canal at 1550 cfs			
	WSE	Channel Invert	Mean Channel Velocity	Flow Area	WSE	Channel Invert	Mean Channel Velocity	Flow Area
	Ft, NGVD	Ft, NGVD	fps	Sq ft	Ft, NGVD	Ft, NGVD	fps	Sq ft
7900	14.61	0.0	2.55	607.58	10.94	-5.0	1.87	826.89
6000	14.04	0.0	2.75	562.87	10.73	-5.0	1.91	809.87
4000	13.26	0.0	3.07	505.60	10.51	-5.0	1.96	791.03
2000	12.15	0.0	3.61	429.15	10.26	-5.0	2.01	771.13
0	10.00	0.0	5.09	304.55	10.00	-5.0	2.07	750.0

C-626 FEB Seepage Collection Canal

The function of the perimeter canal is for seepage collection from the FEB. A series of small agricultural canals currently border the FEB configuration and will be improved to match the current design canal template. Analysis of necessary improvements will be conducted upon receipt of survey data during PED phase. The new perimeter canal will capture seepage and route to a seepage pump station near the southwest corner of the FEB. The design seepage rate is 387 ft^3/day/ft of levee at normal pool depth of 4 feet. A factor of safety of 1.5 was applied, producing a rate of 580.9 ft^3/day/ft of levee, or 0.0067 cfs/ft of levee. The total length of the seepage canal is 11 miles (58,080 ft), which translates into a maximum seepage rate of approximately 390 cfs.

In concurrence with common local agricultural operating procedures, the perimeter seepage canals will be controlled approximately 2.0 ft below average grade in the FEB, at elevation 7.0 ft NGVD. The canal has a bottom elevation of -5.5 ft NGVD, bottom width of 15 ft, side slopes of 1V:2H, and a depth of 14.5 feet. A seepage analysis has been conducted by the USACE Geotechnical Engineering Branch (EN-G) to verify the bottom depth for appropriateness.

TABLE A-10. C-626 SEEPAGE COLLECTION CANAL

Length	Side Slope L/R (S/N)	Canal Bottom Width	Canal Bottom Elevation	Left bank (south/east) elevation	Right bank (north/west) elevation
Miles	1V on ?H	Feet	Ft, NGVD	Ft, NGVD	Ft, NGVD
11.0	1:2/1:2	15.0	-5.50	9.00	9.00

TABLE A-11. C-626 SEEPAGE COLLECTION CANAL SUMMARY OF HYDRAULIC DESIGN DATA (SEEPAGE FLOW)

Station	Channel Design Flow	Mean Channel Velocity
	Cfs	fps
RS 0	391.46	0.74
RS 14500	292.19	0.57
RS 29000	196.47	0.39
RS 43500	95.09	0.19
RS 58000	5.0	0.01

A.5.3.3.2.1.4 Pump Stations

S-626 Seepage Collection Pump

The S-626 structure is the seepage return pump for the A-2 FEB with a total pumping capacity of 700 cfs (500 cfs to accommodate seepage requirements and 200 cfs for design redundancy). The pump station is designed to pump seepage captured in the C-626 Canal. The seepage pump station will be equipped with two 150 cfs electric motor driven pumps and two 200 cfs diesel engine driven pumps that can be used as an alternative method of pumping during commercial power outages or when electric power has high peak demands. The diesel engine drive pumps can also provide additional capacity for high-flow events. Although the peak discharge through the seepage canal is approximately 400 cfs, two 150 cfs electric motor driven pumps were chosen due to the infrequency of higher FEB stages as determined by RSM-BN modeling results for Alternative 4R2 (**Figure A-3**: Depth Exceedance Plot). During times when 400 cfs is needed, a combination of electric motor driven pumps and diesel engine driven pumps can be utilized to meet those needs. The pump station will discharge into the C-625W to be conveyed to the headwater of G-372 pump station. Use of the G-372 seepage pump with an existing design capacity of 150 cfs may be considered as a potential option to reduce the size of the S-626 pump station. Further analysis and optimization of the design will be conducted during PED.

Pump Rates

The S-626 pump station will return seepage intercepted in the FEB seepage canal back to the existing G-372 pump station. The pumping rate was determined based on a seepage rate provided by EN-G, with a factor of safety of 1.5 applied to determine an adjusted seepage rate of 580.9 ft^3/day/ft of levee. The rate was applied to the total linear length of the perimeter levee and used to estimate seepage inflow rates into a Hydrologic Engineering Centers River Analysis System (HEC-RAS) model. The peak seepage inflow resulting from the model established the max pumping rate.

Pump Mix

Pump mixes for seepage management operations were based on a minimum of two bay pump stations to minimize risk of impact to private lands should a single pump fail during critical times. The seepage pump station will be equipped with two 150 cfs electric motor driven pumps and two 200 cfs diesel engine driven pumps. The two diesel engine driven pumps were sized to accommodate the required seepage rate of 400 cfs, and will serve as an alternative power source in cases of power outages and provide additional capacity for infrequent high flow events. Two diesel engine driven pumps are required per SFWMD Major Pumping Station Engineering Guidelines. Electric motors offer efficient operation on a 24/7 basis, such as seepage management. Diesel engines are efficient at stop-go and irregular operations. One criterion for pump mixes was to utilize duplicate pump sizes to reduce overall operation and maintenance costs (reduction in spare parts and focusing of mechanical expertise). Another criterion was to provide a pump mix that allows a smooth pump rate change interval from start-up to full capacity.

Pump Stages

Pump stages as presented in the hydraulic design data sheet were defined by the following pumping parameters:

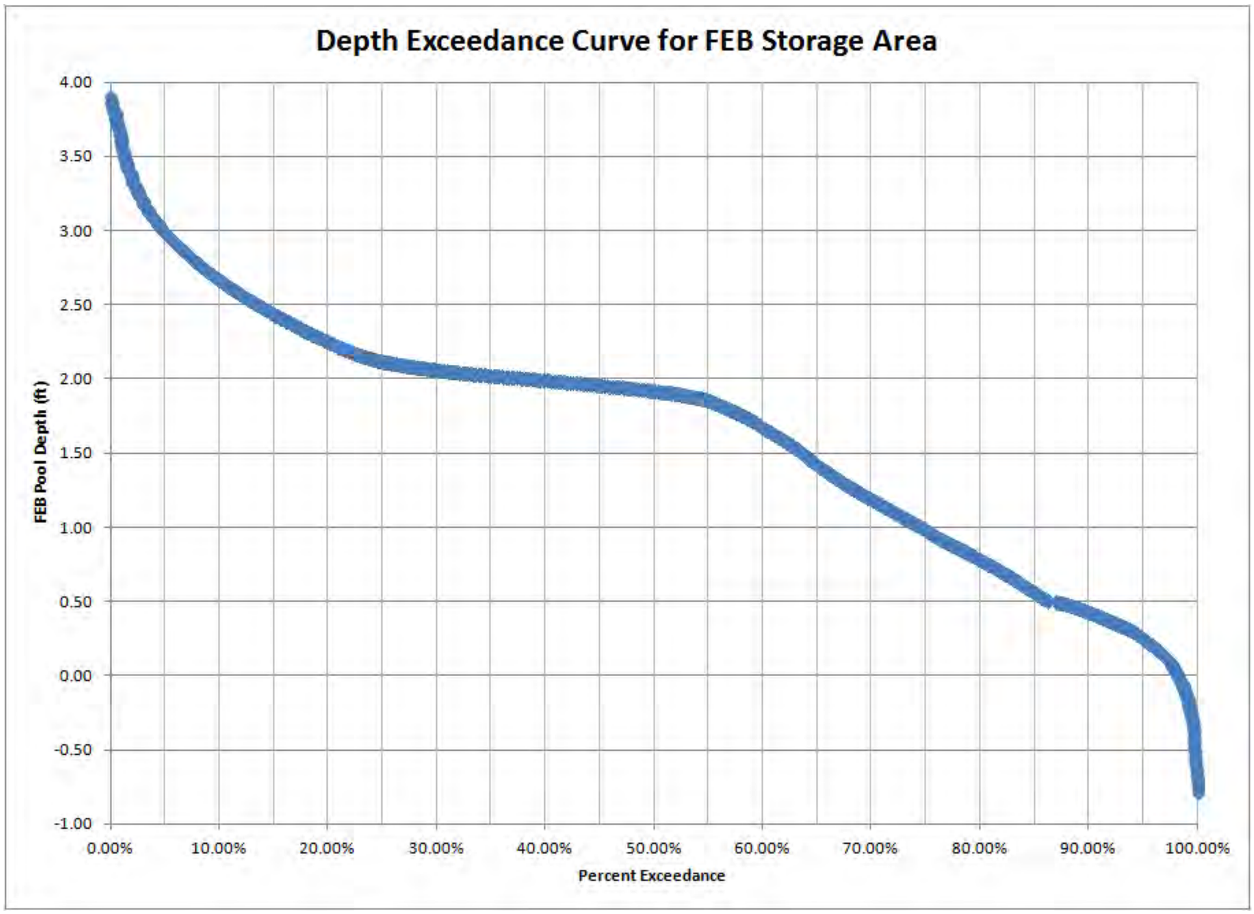
Intake Water Surface Elevations:

Maximum Non-Pumping: Highest canal or pool stage that can be expected to occur.

Maximum Pumping: Maximum canal or pool stage that can be pumped with any increase in stage requiring the pump to be turned off. In most cases, Maximum Non-Pumping and Maximum Pumping stages are identical.
Start Pumping: Canal or pool stage when pump may be turned on as defined by system conditions, typically on the increasing limb.
Normal Drawdown: Expected local drawdown at the pump station intake.
Minimum Drawdown: Lowest local drawdown stage before pump is required to be turned off.
Minimum Non-Pumping: Lowest canal or pool stage that can be expected to occur under non-pumping conditions.

Discharge Water Surface Elevations:
Maximum Non-Pumping: Highest canal or pool stage that can be expected to occur.
Maximum Pumping: Maximum canal or pool stage that can be pumped to, pump is subsequently turned off until stage decreases.
Normal Pumping: Expected normal pool elevations for impoundments and design tailwater stages for conveyance canal pump stations (flood damage reduction drainage discharge).
Minimum Pumping: Lowest canal or pool stage expected when pump may be turned on.
Minimum Non-Pumping: Lowest canal or pool stage that can be expected to occur under non-pumping conditions. In most cases, Minimum Pumping and Minimum Non-Pumping elevations are identical.

FIGURE A-3. FEB DEPTH EXCEEDANCE PLOT FOR RECOMMENDED PLAN ALTERNATIVE 4R2



A.5.3.3.2.1.5 Other Features

<u>S-627 Emergency Overflow Spillway</u>

An orifice-type spillway will provide uncontrolled discharge from the A-2 FEB during extreme events, when FEB discharges are required to protect the embankment integrity. The spillway will include a 265 foot long weir with crest elevation set at 13.50 ft NGVD, or 4.5 feet above the average natural ground of 9.00 ft NGVD within the A-2 FEB. The spillway will discharge into the adjacent seepage canal along the northern portions of the A-1 and A-2 FEBs. The spillway will be located in line with the northern extent of the eastern perimeter levee, adjacent to structure S-628.

To determine the weir length, an unsteady HEC-RAS model was ran using design criteria from ER 1110-2-8(FR) and DCMs 1, 2, and 3. Based on DCM-1, the FEB was determined to have a low hazard potential classification (HPC). For Low HPC, DCM-2 requires the routing of the 100-yr 24-hr storm plus 60 mph wind applied at the peak surcharge stage. DCM-3 states that the SFWMD Basis of Review for Environmental Resource Applications extends the basin permitted rate (storm implicit) to a 100-yr storm level to reduce the potential for localized basin impacts resultant from flood control releases. Under pre-project conditions, localized impacts due to flooding within the EAA basin may potentially occur when extreme rainfall amounts flood a localized area and the water remains in the localized area while not exasperating conditions elsewhere within the EAA through drainage connectivity. Under these pre-project conditions, the flood water remains on-site until downstream conditions improve, whereby the hydraulic gradient finally reaches the site of interest and the basin begins to drain. However, with reservoir overflow spillways, water is conveyed (or spilled) "on top" of the lands already flooded and may potentially be interpreted as exasperating flood conditions. To provide clear guidance regarding what specific criteria may be construed as exasperation of flooded conditions which diminish the level of service of flood protection within a CERP reservoir basin, DCM-3 was drawn up to meet the non-Federal sponsor concerns and reflect USACE's stewardship of public safety. Within the EAA, strict interpretation of DCM-3 would most likely lead to incidental improvement of the level of service of flood protection beyond that for urban areas rather than offsetting potential impacts associated with the CEPP features, at a greater project cost. The USACE hydraulic design rationale to the proposed spillway design is provided in the following bulleted list:

- For the EAA, the (Environmental Resource Permit) ERP basin rule is 20 cfs/sq. mile (CSM) for the 5-yr (assumed 72-hr) storm event. Extending the discharge rate to the 100-yr 72-hr storm is above the DCM-2 requirement for a Low HPC impoundment/reservoir storm routing that is the 100-yr 24-hr storm. The impact of this storm-discharge extension is it would most likely lead to a higher embankment than what most would expect for a Low HPC impoundment.

- In urban areas, the ERP rule is usually near the 20 CSM discharge rate, but, it is typically combined with the 25-yr 72-hr storm event versus the 5-yr storm. Therefore, extending the ERP rule to the 100-yr 72-hr rate would provide better protection from a potential for impact than for urban areas, again, at a greater project cost.

- Since the 5-yr 72-hr storm has a total rainfall of 7 inches and the crest elevation is to be set at 6 inches above the Normal Full Storage Level (NFSL), then the impoundment will capture all but 1 inch of the total design storm depth. The additional 6 inches above the normal spillway crest setting at NFSL is to prevent overtaxing of the seepage management system with more common frequent storm events since the spillway does not directly discharge into an adjacent major canal.

- The ERP 20 CSM rule provides for a discharge of 437.5 cfs, but 1 inch head on the proposed 265 foot spillway has a discharge rate of only 19.1 cfs (see **Table A-12** for storm routing summary). By proposing to extend the 5-yr 72-hr to the 100-yr 24-hr event, the discharge nearly equates to the ERP rule with 443.4 cfs (6 cfs more).

- The use of the 100-yr 24-yr sized 265 foot long spillway allows a 3 foot freeboard on the USACE historically required 50% PMP surcharged pool peak stage on the Low HPC impoundment with the proposed minimal 9 foot embankment (nearly so, 2.85 foot freeboard actually). This freeboard lowers risk of breach with extreme storm events (50% PMP equates with 27 inches of depth).

Based on consideration of this collective rationale, USACE Jacksonville District Engineering Division believes that this minor variance from DCM-3 is maintaining the original DCM intent and demonstrates an optimal design that meets all agency requirements as intended. Since the FEB stages over the RSM-BN simulated period of record do not overtop the FEB emergency spillway, the S-627 emergency overflow spillway design details, including discharge location, were not further considered during the CEPP Savings Clause evaluation of the recommended plan (detailed in Annex B of the PIR). A more detailed flood routing using the FEB seepage collection system (including the A-1 and A-2 seepage canals) will be conducted during PED to ensure there will be no additional adverse impacts to adjacent agricultural lands. During PED, the seepage system will be refined to ensure sufficient capacity to capture any additional discharges from S-627. It is anticipated that if necessary, water will be routed through the S-8 pump station for discharge into the Miami Canal, as it currently serves as a flood control structure for the EAA today.

TABLE A-12. EMERGENCY OVERFLOW SPILLWAY ANALYSIS

Storm	Discharge Criteria	ERP Flow rate (cfs)	Crest Elev. (ft, NGVD)	Crest Length (ft)	Max Stage (ft, NGVD)	Max Head, weir (ft)	Max Depth (ft)	Max Flow (cfs)
100-yr 24-hr	3/4"/day	440	13	110	14.13	1.13	5.13	437.01
			13.5	265	14.14	0.64	5.14	443.42
100-yr 72-hr	3/4"/day	440	13	70	14.54	1.54	5.54	442.92
			13.5	125	14.56	1.06	5.56	447.8
			13.5	265	14.52	1.02	5.52	815.38
Previous 50%, 72-hr PMP			13.5	1,500	15.03	1.53	6.03	3,007.42
			13.5	265	15.15	1.65	6.15	1,845.57

A.5.3.3.2.2 Existing Structures

<u>G-370 Pump Station</u>

G-370 is an existing pump station that is currently being used to deliver water from the North New River Canal to the STA 3/4 inflow structure. Stormwater runoff and Lake Okeechobee releases in the North New River Canal are pumped into the STA 3/4 distribution system. Prior to CEPP implementation, this pump station will also be used as one of the A-1 FEB's inflow structures. This is a six bay pump station with three 925 cfs diesel pumps and three 75 cfs electric driven seepage pumps. The total flood control capacity is 2,775 cfs. The pump station is located at the southwestern end of A-1 adjacent to US-27 and the North New River Canal.

<u>G-372 Pump Station</u>

G-372 is an existing pump station that is currently being used to deliver water from the Miami Canal to the STA 3/4 inflow structure. Stormwater runoff and Lake Okeechobee releases in the Miami Canal pass through this pump station and into the STA 3/4 distribution system. Prior to CEPP implementation, this pump station will also be used as one of the A-1 FEB's inflow structures. For CEPP, this pump station will also be used as the A-2 FEB's inflow. This is a seven bay pump station with four 925 cfs diesel pumps and three 75 cfs electric drive seepage pumps. The total flood control capacity is 3,700 cfs. For CEPP purposes, the capacity of G-372 will be reduced to 1,550 cfs when used for inflow into the FEB. The pump will operate at normal capacity operations when providing conveyance through the STA 3/4 Supply Canal.

<u>S-7 Pump Station</u>

S-7 is an existing pump station that is currently used to discharge runoff water via the North New River Canal, as well as provide an outlet for STA 3/4 discharges, into WCA-2A. The pump station is equipped with three 830 cfs diesel pumps for a total capacity of 2,490 cfs. For CEPP, the majority of STA 3/4 discharges will be delivered to the modified S-8 Pump Station for delivery to WCA-3A, and operation of the S-7 Pump Station will likely be limited to peak events. The pump station is located in the alignment of the North New River Canal at the northwestern corner of WCA-2A.

<u>S-8 Pump Station</u>

S-8 is an existing pump station that is currently used to discharge runoff water via the Miami Canal, as well as provide an outlet for STA 3/4 discharges, into WCA-3A. CEPP will maintain this existing design capacity for the S-8 complex through a combination of the following design considerations: pump station design modifications, a new hydraulic connection from S-8 to the degraded L-4 Levee (New S-8A), utilization of the existing G-404 pump station (570 cfs design capacity), and leaving the 1-2 mile segment of the Miami Canal as available getaway conveyance capacity during peak flow events. For CEPP, the S-8 pump station and/or G-404 may require design modifications (or possible replacement). The Recommended Plan cost estimate includes costs for the potential S-8 complex modifications, which are included as the new S-8A (canal connection to L-4 and two culverts structures). During PED, the following design uncertainties will be assessed/reassessed in further detail: modifications to S-8 and/or G-404, to address pump efficiency concerns; the proposed S-8A culvert and associated canal connecting the Miami Canal to the L-4 Canal; and the required length of the unmodified Miami Canal to maintain hydraulic getaway conveyance capacity. Flood control operation capability will be maintained during S-8 modification construction. S-8 is equipped with four 1,040 cfs diesel pumps for a total capacity of 4,160 cfs. The pump station is located in the alignment of the Miami Canal at the northern boundary of WCA-3A.

A.5.3.3.2.3 Existing Canals

<u>Miami Canal</u>

EAA runoff and Lake Okeechobee regulatory releases to the Miami Canal will be captured by the existing G-372 pump station and distributed to the A-2 FEB when operational criteria is met, including FEB stage constraints for additional inflows and availability of STA treatment capacity. Previous analysis of the existing Miami Canal conditions indicated the maximum in-bank conveyance capacity was 1,550 cfs when pulling from Lake Okeechobee during storm off-peak times. It was determined that no improvements to the Miami Canal template would be necessary and all CEPP components north of the redline receiving flow from the Miami Canal would be sized to accommodate 1,550 cfs.

<u>North New River Canal</u>

EAA runoff and Lake Okeechobee regulatory releases to the North New River Canal (NNR) will be captured by the existing G-370 pump station and distributed to the A-1 FEB when operational criteria is met, including FEB stage constraints for additional inflows and availability of STA treatment capacity. Previous analysis of the existing NNR Canal conditions indicated the maximum in-bank conveyance capacity was 1,350 cfs when pulling from Lake Okeechobee during storm off-peak times. It was determined that no improvements to the NNR would be necessary and all CEPP components north of the redline receiving flow from the NNR Canal would be sized to accommodate 1,350 cfs.

<u>STA 3/4 Supply Canal</u>

The STA 3/4 Supply Canal conveys discharge from G-370 and G-372 pump stations to the STA 3/4 intake structures. The Supply Canal is located adjacent to the northern boundary of the Holey Land Wildlife Management Area and extends from G-372 eastward (7.7 miles) and then southward (2.7 miles) before intersecting the STA 3/4 Inflow Canal at the northwest corner of the STA. For CEPP, the STA 3/4 Supply Canal will provide the inflow route for the A-2 FEB via S-624, as well as convey flows discharged from the A-2 FEB to STA 3/4 via S-625.

STA 3/4 Seepage Canal

The STA 3/4 Seepage Collection Canal is located along the northern boundaries of the STA 3/4 and the Supply Canal. Its purpose is to collect seepage and convey water to the G-370 and G-372 pump stations at the east and west ends of the canal, respectively. The segment of the Seepage Canal along the southern boundary of the A-2 FEB will be utilized in its existing condition as a collection canal to direct flows in the FEB to the S-625 discharge structure.

A.5.3.3.2.4 Embankments

A.5.3.3.2.4.1 Embankment Height Analysis

The FEB at this time carries a low hazard potential classification (HPC) per DCM-1, which is extended to embankment design. Embankment top widths are 14 feet wide per DCM-4, with dam heights determined by using the greater height based on analysis of the following criteria (ER-1110-8-2(FR), ER-1110-2-1156, DCM-2, and risk):

1. Three feet above the maximum surcharge pool elevation. The maximum surcharge pool elevation is based on the greatest elevation resulting from the following storm routings:
    a. The Inflow Design Flood (IDF), which is identified as the 100-yr 24-hr storm event for the CEPP FEB, per DCM-2;
    b. The 50% 72-hr PMP per ER-1110-8-2(FR); and

Wind setup and wave run-up analysis on critical fetch lengths with the impoundment at full pool. Wave run-up is dependent on levee slope (steeper slopes have higher run-up).

The storm routing analyses conducted for the emergency overflow spillway (section A.5.3.3.2.1.5 Other Features) were integral to the development of embankment heights. The results of the storm routings were used to determine the maximum surcharge stage. Per Table A-12, based on the selected weir length of 265 feet, the maximum surcharge stage resulted from the 50% 72-hr PMP storm at an elevation of 15.15 ft NGVD. Per ER-1110-8-2(FR), the minimum levee height should be 18.15 ft NGVD.

Wind setup and wave run-up analysis evaluated freeboard requirements for differing embankment types (earth, riprap) at varying slopes (1v:3h and 1v:4h) for the 100-yr 24-hr storm and the 50%, 72-hr PMP. Overtopping (over wash) of the earthen embankments is permitted to a maximum rate of 0.1 cfs/ft. In each scenario, the resulting freeboard is less than three feet (maximum is 2.3 ft). Therefore, the recommended freeboard is dictated by the minimum requirement for a Low HPC basin (3.0 feet) rather than the wind and wave conditions for the site. Refer to Appendix A, Annex A-1: Hydraulic Design, section A.3.8 for complete FEB Wind/Wave Analysis.

Preliminary embankment analysis recommended an embankment crest elevation of 20.3 feet NGVD, based on available data at the time. All hydraulic analysis will be revised during PED to incorporate the revised recommended elevation of 18.00 ft NGVD (rounded from 18.15 ft NGVD).

A.5.3.4 Risk and Uncertainty

This section presents qualitatively the risk and uncertainty associated with the project as designed for this PIR. Understanding that the current USACE philosophical approach to Feasibility Studies is to be quick and limit analyses to that for benefit and cost determinations, acknowledging risk and uncertainty in the hydraulic design of the project will be an important part of the risk registry. The overall approach to the hydraulic design was to be conservative enough to capture expected costs without being unrealistic in overestimation, yet not to underestimate beyond what optimization and the savings that could be realized during PED phase efforts.

A.5.3.4.1 Hydrologic and Hydraulic Computer Software Tools

Several hydrologic and hydraulic computer software tools were utilized in the formulation of alternatives and the Recommended Plan. Interpretation of hydraulic design results should consider the inherent strengths and limitations of the underlying hydrologic and hydraulic tools. Additional descriptions of the modeling tools are provided in Appendix A, Section A.8.1 (Modeling Strategy).

A.5.3.4.2 Flow Equalization Basin

<u>Low Hazard Potential Classification</u>
This section discusses the risk imposed by the Flow Equalization Basin (FEB) feature during precipitation and wind storm events with current freeboard design and proposed operations. To establish how risk is to be viewed, i.e. consequences, it is necessary to first review the potential hazard that may be posed by the constructed project. The FEB carries a Low Hazard Potential Classification (Low HPC) for all the reasons listed in the following:

(1) No expected loss of life should a breach occur.

No residents nearby.

No businesses or institutions nearby.

No highways or evacuation routes at risk (US-27 nearest distance is 2.5 miles. However, the road crest elevation is 3.3 feet above FEB max pool level (50% PMP):

[18.5 crest el – (9 ground el +6.15ft deep) = 3.35ft].

(2) No high-value private properties are at risk, as the area is primarily agriculture with several quarries. Northern and western perimeter embankments are the only embankments adjacent to private lands, all agricultural. Southern perimeter embankment is adjacent to a large conveyance canal with a southern raised bank that can function as a secondary containment embankment, further south of which is natural, but disturbed wetlands (Holey Lands). Eastern perimeter embankment is adjacent to the proposed FEB to be constructed on lands formerly as the A-1 cell. Until then, this same land is non-developed former agriculture lands.

(3) Except for direct rainfall, inflow is controlled (i.e., pump inflow only)

(4) Potential volume of water storage is large because of areal size of the project; however the surrounding agricultural area is proportionally vast as well, i.e. spread of water limits depth in immediate vicinity of the FEB. Also, with primary crop being tall and thick sugar cane, the spread of breach flow will be at low velocities.

<u>Freeboard under Design Conditions</u>
The current FEB freeboard design is between 5.5 ft and 5.2 ft as defined by the vertical height between the IDF (100-yr and 50%PMP, respectively) pool and embankment crest elevation (normal full pool of 4 ft depth) at water surface elevation 13.00 ft NGVD. To evaluate the appropriateness of this freeboard, **Table A-13** provides minimum freeboard requirement based on historic Federal requirement and current DCM-2 joint USACE and SFWMD requirements.

TABLE A-13. EMBANKMENT RISK

Policy	Precip. Event	Starting Pool Elev.	Routed IDF	Wind Event	Setup	Full Run Up	Overwash 0.1cfs	Min. 3-foot	Min Embk Crest El
DCM-2	100-yr, 24-hr	el. 13.0	+1.14ft el. 14.14	60mph	+2.40ft el. 16.54	+0.20	el. 16.60 (rnded)	el. 17.14	el. 17.78
USACE ER 1110-8-2	50% PMP, 72-hr	el. 13.0	+2.15ft el. 15.15	60mph*	+2.09ft el. 17.24	+0.24	el. 17.30 (rnded)	el. 18.15	el. 18.00

Note: * denotes not defined, but what was assumed. All elevations are referenced to NGVD 29.

1. Stages, setup, run up, and overwash are not identical to that provided in wind-wave stand-alone report located in the Appendix A, Annex A-1, but are approximations based on same report for modified design stages. The result of minimum required embankment height ultimately remains the same. Absolute numbers for modified stages will be provided in DDR on authorization.
2. This table will be updated in PED to provide accurate numbers versus approximations as provided here.

Current design crest elevation is set at 20.30 ft NGVD (based on available data at the time of design; will be updated during PED), or 11.3 ft above average natural ground of 9.00 ft NGVD. The very low wave run up is due to emergent vegetation being the primary cover, as in the case of SFWMD stormwater treatment areas (STA) and other shallow storm water impoundments. For the FEB with the high loading of phosphorous expected, cattails will likely be the prominent species, which are characteristically tall in excess of 8 feet and typically dense, with shedding of elongated leaves. These characteristics reduce shear stresses between the air-water interface that leads to wave generation/growth. Additionally, the actual wind setup is not expected to be as high as modeled because of the wave mitigating effects of vegetation growth. Therefore, the embankment is of sufficient height to minimize risk of structure loss due to wave overtopping/overwash activity in accordance to the Low HPC assigned to the structure and established policies, including the current ER 1110-2-1156.

A.5.3.5 HYDRAULIC DESIGN DATA SHEETS

TABLE A-14. S-623 GATED SPILLWAY
Hydraulic Design Data Sheet

Location STA 3/4 Supply Canal, approx. 800 feet upstream of G-372; x= 718,541 y = 764,032

Purpose S-623 is a divide structure between Miami Canal water and FEB discharges.

Design Conditions	Discharge	3,700	cfs
	Headwater Elevation	10.25	feet, NGVD 29
	Tailwater Elevation	10.00	feet, NGVD 29
Crest Data	Shape	Ogee	
	Design Head (Hd)	6.75	feet
	Net Crest Length	140.0	feet
	Crest Elevation	3.50	feet, NGVD 29
	Approach Apron Elevation	-2.00	feet, NGVD 29
	Weir Control	Vertical Slide	
Gates	Number of Gates	4	
	Gate Width	35.0	feet
	Gate Height	14.0	feet
	Clearance Elevation	15.00	feet, NGVD 29
	Breastwall Elevation	TBD	feet, NGVD 29
	Intermediate Pier Width	3.25	feet
Stilling Basin	Design Discharge	3,700	cfs
	Apron Elevation	-2.00	feet, NGVD 29
	Apron Length/Width	36.0/149.75	
	End Sill Elevation	-1.50	feet, NGVD 29
	Top of Baffle Block Elevation	0.00	feet, NGVD 29
	Dist from crest toe to 1st row of blocks/2nd row	18.00	feet
	Velocity over End Sill	2.15	fps
	Training Wall Elevation	TBD	feet, NGVD 29
Canal Data (US/DS)	Invert - Thalweg	-13.5	feet, NGVD 29
	Top of Bank (E/W)	15.5/12.0	feet, NGVD 29
	Bottom Width	150.0	feet
	Top Width	252.0	feet
	Side Slope (V:H)	1 on 2	
Revetment	Riprap Extent (Downstream)	TBD	feet
	Riprap Size (D50)	TBD	feet
	Riprap Specific Weight	TBD	lb/ft3
	Max Velocity Riprap Can Withstand	TBD	fps

TABLE A-15. S-624 GATED CULVERT

Hydraulic Design Data Sheet

Location STA 3/4 Supply Canal, approx. 1.5 miles east of G-372; x = 727,613 y = 764,144

Purpose S-624 is a fully submerged gated sag culvert that conveys flow from G-372 into the A-2 FEB. The culvert invert is set such that it is below natural grade elevation. When closed, obstructs flow from G-372 from entering the FEB to allow flow through the STA 3/4 Supply Canal.

Design Conditions	Discharge	1,550	cfs
	Headwater Elevation	16.75	feet, NGVD 29
	Tailwater Elevation	14.25	feet, NGVD 29
Culvert Data	Number of Barrels	2	
	Barrel Type	Concrete Box Culvert	
	Box Width	11	feet
	Box Height	11	feet
	Culvert Length	350	feet
	Upstream Invert	-4.50	feet, NGVD 29
	Downstream Invert	-4.50	feet, NGVD 29
	Number of Bends	4	
	Bend Angle	45	degrees
	Sag Invert Elevation	-14.00	feet, NGVD 29
	Natural Water Table	9.00	feet, NGVD 29
	Headwall - HW Elevation	TBD	feet, NGVD 29
	Headwall - TW Elevation	TBD	feet, NGVD 29
	Wingwall - HW Elevation	TBD	feet, NGVD 29
	Wingwall - TW Elevation	TBD	feet, NGVD 29
Canal Data	Side Slopes (V:H)	1 on 2	
	Upstream Bottom Width	40	feet
	Upstream Bottom Elevation	-5.00	feet, NGVD 29
	Downstream Bottom Width	40	
	Downstream Bottom Elevation	0.00	feet, NGVD 29
Energy Dissipation	**Riprap Requirements**		
	Rip Rap Design Velocity	6.75	fps
	Upstream Length	TBD	feet
	Upstream Protection Elevation	TBD	feet, NGVD 29
	Downstream Length	TBD	feet
	Downstream Protection Elevation	TBD	feet, NGVD 29

TABLE A-16. S-625 GATED CULVERT

Hydraulic Design Data Sheet

Location A-2 FEB western perimeter levee; x= 726,458 y = 764,793

Purpose S-625 conveys flows from the A-2 FEB to the G-372 pump station via a new discharge canal (C-625W) from the FEB. S-625 will only discharge into G-372 if flow from the Miami Canal is blocked via use of S-623 gated spillway.

Design Conditions	Discharge	1,550	cfs
	Headwater Elevation	12.00	feet, NGVD 29
	Tailwater Elevation	11.00	feet, NGVD 29
Culvert Data	Number of Barrels	3	
	Barrel Type	Concrete Box Culvert	
	Box Width	9	feet
	Box Height	9	feet
	Culvert Length	140	feet
	Upstream Invert	0.50	feet, NGVD 29
	Downstream Invert	0.50	feet, NGVD 29
	Natural Grade	9.00	feet, NGVD 29
	Natural Water Table	9.00	feet, NGVD 29
	Headwall - HW Elevation	TBD	feet, NGVD 29
	Headwall - TW Elevation	TBD	feet, NGVD 29
	Wingwall - HW Elevation	TBD	feet, NGVD 29
	Wingwall - TW Elevation	TBD	feet, NGVD 29
Canal Data	Side Slopes (V:H)	1 on 2	
	Upstream Bottom Width	45	feet
	Upstream Bottom Elevation	0.00	feet, NGVD 29
	Downstream Bottom Width	45	
	Downstream Bottom Elevation	-5.00	feet, NGVD 29
	DS Slope from Invert to Canal Bottom	1 on 5	
Energy Dissipation	**Riprap Requirements**		
	Rip Rap Design Velocity	6.5	fps
	Upstream Length	TBD	feet
	Upstream Protection Elevation	TBD	feet, NGVD 29
	Downstream Length	TBD	feet
	Downstream Protection Elevation	TBD	feet, NGVD 29

TABLE A-17. S-626 PUMP STATION
Hydraulic Design Data Sheet

Location Western boundary of the A-2 FEB, north of the S-625 discharge structure.

Purpose/Operational Intent: Seepage Control, Non-Flood Control
Provides seepage control for the A-2 FEB impoundment by the backpumping of collected seepage from the FEB into G-372 via C-625W Discharge Canal.

Design Condition:		500	cfs
Design Capacity:		700	cfs
Pump Station Capacity Criteria:			

The design pump rate was determined by seepage rate analysis and incorporating a safety factor of 1.5.

Number of Pumps		4	
Pump Mix Type and Size	Electric	2@ 150	cfs
	Diesel	2@ 200	cfs

Mix Criteria:

1. The pump station will have 4 bays; two identical 150 cfs electric motor driven pumps, and two 200 cfs diesel engine driven pumps
2. The pump mix allows for increased capacity during peak storm events, while having duplicate pumps throughout the system for operation and maintenance consideration

Control	TBD	
Design Heads		
Normal (HW=13.0 NGVD, TW=7.0 NGVD)	6.0	ft
Maximum (HW=15.15 NGVD, TW=7.0 NGVD)	8.15	ft
Intake Water Surface Elevations		
Maximum Non-Pumping Pumping	TBD	ft, NGVD
Maximum Pumping	TBD	ft, NGVD
Start Pumping	TBD	ft, NGVD
Normal Drawdown	TBD	ft, NGVD
Minimum Drawdown Pumping	TBD	ft, NGVD
Minimum Non-Pumping	TBD	ft, NGVD
Channel Invert	-5.50	ft, NGVD
Discharge Water Surface Elevations		
Maximum Non-Pumping	TBD	ft, NGVD
Maximum Pumping	TBD	ft, NGVD
Normal Pumping	TBD	ft, NGVD
Minimum Pumping	TBD	ft, NGVD
Minimum Non-Pumping	TBD	ft, NGVD
Channel Invert	-5.50	ft, NGVD

TABLE A-18. S-627 EMERGENCY OVERFLOW SPILLWAY
Hydraulic Design Data Sheet

Location A-2 FEB northeast corner; adjacent to the S-628 structure

Purpose The S-627 emergency overflow spillway will provide uncontrolled discharge from the A-2 FEB during extreme events into the adjacent seepage canal along the northern perimeter of the A-1 and A-2 FEBs.

Design Conditions	Discharge		1,845	cfs
	Headwater Elevation		15.15	feet, NGVD 29
	Tailwater Elevation		9.00	feet, NGVD 29
Maximum Expected Stages				
	Headwater Elevation		15.25	feet, NGVD 29
	Tailwater Elevation		10.00	feet, NGVD 29
Maximum Head Differential				
	Headwater Elevation		15.25	feet, NGVD 29
	Tailwater Elevation		9.00	feet, NGVD 29
Weir Design Data				
	Weir Type		Broad-crest	
	Crest Elevation		13.50	feet, NGVD 29
	Crest Length		265.00	feet
	Minimum Tieback Elevation		TBD	feet, NGVD 29
	Weir Control		Passive, none	
Canal Data	A-2 FEB			
		Side Slope Cotangent	3	
		Bottom Width	Pool	
		Bottom Elevation	9	feet, NGVD 29
	Combined Seepage/Spillway Conveyance			
		Side Slope Cotangent	2	
		Bottom Width	15	feet
		Bottom Elevation	-5.5	feet, NGVD 29
Apron/Riprap Requirements - to be verified with final geotechnical design				
	Apron Length		30	feet
	Minimum Riprap Size		TBD	feet

TABLE A-19. S-628 GATED CULVERT
Hydraulic Design Data Sheet

Location	Northeast corner of A-2 FEB on eastern perimeter; x = 757,854 y = 778,977		
Purpose	Conveys flows from A-2 FEB into A-1 FEB, or vise versa, from A-1 into A-2.		
Design Conditions	Discharge	930	cfs
	Headwater Elevation	12.5	feet, NGVD 29
	Tailwater Elevation	11.5	feet, NGVD 29
Culvert Data	Number of Barrels	2	
	Barrel Type	Concrete Box Culvert	
	Box Width	9	feet
	Box Height	9	feet
	Culvert Length	140	feet
	Upstream Invert	1.50	feet, NGVD 29
	Downstream Invert	1.50	feet, NGVD 29
	Natural Grade	9.00	feet, NGVD 29
	Natural Water Table	9.00	feet, NGVD 29
	Headwall - HW Elevation	TBD	feet, NGVD 29
	Headwall - TW Elevation	TBD	feet, NGVD 29
	Wingwall - HW Elevation	TBD	feet, NGVD 29
	Wingwall - TW Elevation	TBD	feet, NGVD 29
Canal Data	Side Slopes (V:H)	1 on 2	
	Upstream Bottom Width	35	feet
	Upstream Bottom Elevation	1.50	feet, NGVD 29
	Downstream Bottom Width	35	
	Downstream Bottom Elevation	1.50	feet, NGVD 29
	Slope from invert to natural grade (V:H)	1 on 5	
Energy Dissipation	**Riprap Requirements**		
	Rip Rap Design Velocity	5.8	fps
	Upstream Length	TBD	feet
	Upstream Protection Elevation	TBD	feet, NGVD 29
	Downstream Length	TBD	feet
	Downstream Protection Elevation	TBD	feet, NGVD 29

A.5.4 STRUCTURAL DESIGN

A.5.4.1 General Status of Completed and Non-Executed Efforts

Structural design of S-623, S-624, S-625, S-626, S-627 and S-628 will be completed during the design phase. During design phase the structural calculations will be completed after survey, hydraulic design, and geotechnical investigations are performed. The structural design will conform with the appropriate Engineering Manuals (EM), Engineering Regulations (ER), or Design Criteria Memorandums (DCM).

A.5.4.2 Pumping Stations

S-626 is a seepage pump station that will be similar in design to S-357, but with the new layout of the Miller (S-488) pump station.

A.5.4.3 Overflow Spillways

S-623 is a gated structure similar to S-65EX1, using a two-phased approach and offsetting the structure will not require a bypass canal to be designed for construction of the structure.

A.5.4.4 Culverts

S-624, S-625, and S-628 are gated box culverts that will be designed similar to the (S-276 (C-4A)) culverts on Herbert Hover Dike (HHD).

A.5.4.5 Weirs

S-627 is an overflow weir that will have the same crest width as the levee of 14 feet. The design will be similar to the overflow weir design in the C-111 South Dade S-327.

A.5.5 MECHANICAL AND ELECTRICAL DESIGN

A.5.5.1 General

The pumping station mechanical design shall be in accordance with Hydraulic Institute Standards, EM 1110-2-3102 (General Principles of Pumping Station Design and Layout), and EM 1110-2-3105 (Mechanical and Electrical Design of Pumping Stations). The design will also follow the guidance of ETL 1110-2-313 (Hydraulic Design Guidance for Rectangular Sumps of Small Pumping Stations with Vertical Pumps and Ponded Approaches).

The seepage pumping station will have a required pumping capacity of 500 cfs.

The pump mix will be further developed during the design phase of the project, but it will likely have a mix similar to having two 200-cfs diesel engine driven pumps and two 150-cfs electric motor-driven pumps.

The pump intakes will likely be suction bell type. The use of formed suction intakes at the pumps shall be evaluated during preparation of the plans and specifications for the pumping station and shall be based upon the channel intake design.

Axial flow pumps will be used for the pumping station. The decision on whether the pumps will have either a conventional or siphon discharge will be determined during the preparation of the plans and specifications.

The pumping station electrical design shall be in accordance with NEC, NFPA, IESNA, TIA/IEA, IEEE, and recommended practice. Also, EM 1110-2-3102 (General Principles of Pumping Station Design and Layout) and EM 1110-2-3105 (Mechanical and Electrical Design of Pumping Stations) will be used.

Although the capacity of this station is low enough that SFWMD's Major Pumping Station Engineering Guidelines is not applicable, we will follow the applicable portions of these guidelines.

A.5.5.2 General Status of Completed and Non-Executed Efforts

Mechanical and electrical design of S-623, S-624, S-625, S-626, S-627 and S-628 will be completed during the design phase. During design phase the mechanical and electrical calculations will be completed after survey, hydraulic design, and geotechnical investigations are performed. Conceptual design is developed for use for cost assumptions.

A.5.5.3 Pumping Station S-626

The larger pumps will likely be designed with a suction bell intake. These pumps may be conventional-discharge or siphon-discharge type. Each pump will be driven by a diesel engine through a right-angle reduction gear.

The smaller pumps will be axial-flow-type vertical-shaft pumps. The pumps will be driven by direct-drive electric motors.

The pumps are expected to run at less than 500 rpm with an efficiency of about 80%. The diesel engine pump drives for the 200-cfs pump should be about 600 horsepower each.

The pumping station will include various support items, including the following:

a. Diesel fuel system, including vaulted double-wall aboveground fuel storage tanks capable of holding enough fuel to operate the engine driven pumps and an emergency generator continuously for seven days.
b. Hoisting system for maintenance or repair of the pumping equipment.
c. Toilet facility with a water closet and a lavatory.
d. Kitchen-type sink.
e. Potable water system and a septic system for the plumbing fixtures.
f. Ventilation system to provide fresh air in the pump bays, generator area, and toilet room.
g. Air-conditioning system for the office.
h. Stilling wells containing float switches to be used for pump operations and water level monitoring.

A.5.5.3.1 Pumping Station Features

Pump Drives
The diesel engines will be standard model full-diesel type, 2 or 4 cycle, with mechanical injection and cooling provided by keel coolers. Diesel engine horsepower will be about 600 hp each.

Engine Auxiliary System
Cooling of the engines will be by means of a closed system consisting of keel coolers, overhead expansion tanks, and engine-driven jacket water and aftercooler water circulating pumps with proper heat balance maintained by the thermostatically controlled proportioning valves. The main lubricating oil pump for each engine will be driven directly by the engine.

Speed Reduction Gear
Power will be transmitted from the engines to the pumps by means of right-angle type gear reducers. The units will be designed for an application factor of 2.0 times the maximum input power. Thrust load due to hydraulic unbalance and an anti-friction type bearing located within each reducer unit will carry the weight of pump rotating elements. Connection between each reducer and engine will be by flexible coupling to compensate for misalignment and vibration or shock transmission. Each reducer will be provided with forced lubrication from a direct connected positive displacement pump with cooling of the oil by an external system. To prevent reverse rotation, the transmission would be fitted with an anti-reverse rotation clutch.

Fuel Oil Storage System and Supply
Aboveground Storage Tanks (ASTs) will be located at a safe distance from the station. ASTs shall be concrete-vaulted and have a dual containment feature. Multiple tanks may share the total capacity for the station. Fuel capacity should be for seven days, 24-hour/day continuous operation at maximum fuel consumption rate. The tanks will be filled from fuel trucks. The tanks will be connected to the station supply header. The fuel system for the engines will consist of day tanks (typically up to 250 gallons capacity each) to supply each diesel engine. The day tanks will have automatic operation in sending and receiving fuel and controlling the level of the fuel inside of the day tank. A similar day tank will be provided for the engine generator set.

Station Crane/Hoist
An overhead bridge-type electric crane will be provided. The crane/hoist shall be capable of handling up to 15-ton loads. The crane/hoist will handle pumping station equipment, such as the diesel engines, reduction gears, or pump components during initial installation, as well as for general service thereafter.

Diesel Engine-Generator Sets
A diesel engine-driven generator set with capacities up to 500 kW may be provided. This generator must provide general standby power, but it also may be required to provide sufficient power to operate one of the electric motor driven pumps for as long as seven days.

Potable Water and Plumbing
A potable water supply and plumbing system will be provided. This will include a septic system. A filtered water system will be necessary for the station to supply water to a Toilet (lavatory, shower, and water closet) and small kitchen area.

Air Conditioning
Small split-system air conditioning systems will be provided for the control room, telecommunications room, and the break room.

Ventilation System
A system of air inlet openings and exhaust fans will be provided for ventilation of the operating floor area. The air inlet louvers will be the type commonly referred to as Miami-Dade louvers. Bird screening will also be provided over the openings. The wall type exhaust fans will have motor-operated dampers.

Trash Rake
Trash rake/rack system will be one of two types: an automatic, continuously rolling, flex rake and trash rack system such as that manufactured by Duperon, or a powered rail-mounted traveling trash rake and hoist car assembly with a telescoping arm used to grip and remove debris. This system is similar to ones that are manufactured by Hydro Component Systems. The system selected shall be similar to those that have proven satisfactory at previously completed pumping stations.

Pump Model Tests
The specifications will require that a series of model tests be performed to verify performance and cavitation limits of the proposed pump. The contractor will be required to construct one complete pumping system for each size pump to the necessary scale model. The pumping system will include the forebay, pump, and discharge tube. All tests for determination of compliance with guarantees of capacity and/or efficiency will be accomplished using prototype heads.

A.5.5.3.2 Electrical Features

Electric Service and Backup Generator
A 480-volt, three phase, electrical service shall be provided. The local utility company shall provide the power. The diesel engine-generator unit shall be provided to supply 480-volt, three phase electrical power when utility power is not available or not reliable. Transient Voltage Surge Suppression (TVSS) shall be provided at the service entrance. The backup generator and automatic transfer switch will be sized sufficiently to power diesel engine auxiliaries, trash rakes, exhaust fans, lights and SCADA equipment.

Interior Electrical Distribution
Switchgear rated for 480 volt, three phase with a main breaker will be connected to the incoming service and will feed engine control centers, motor control centers, lighting panels, power panels and station equipment defined in the Pumping Station Features above. Each engine control center will house starters and controls for auxiliary equipment for the engine unit. The main switchboard will also feed transformers to supply 120/208 or 480/277 volt loads as necessary.

Interior and Exterior Lighting
High intensity discharge, industrial high bay luminaries will be used for the main pumping station area with industrial fluorescent fixtures with electronic ballasts for office and general type areas. Exterior lighting for security purposes would be automatically controlled by photo-electric cells and contactors.

Wiring and Conduit
Insulated copper conductors will generally be installed in either PVC coated rigid galvanized steel conduit or schedule 80 rigid plastic conduit. Conductors will be rated for 600 volt insulated types XHHW or XHHW-2. All wiring will conform to UFGS Guide Specifications.

Instrumentation and Controls

The pumping station will have a centralized monitoring and control room. Each diesel engine pump drive will have a separate motor control center to supply power and house controls for the engine auxiliaries, such as jacket water pump, engine lube pump, fuel filter pump, etc. Each diesel engine will also be equipped with a separate instrument panel and will house engine start/stop controls and pressure and temperature indicators to indicate engine performance. Programmable logic controllers (PLCs) will be used to monitor and control the engine and station auxiliaries. An Ethernet network will connect the PLCs and station computer. Ethernet based IP cameras will also connect to the Ethernet network. The station computer will allow for operation of the station via SFWMD's preferred SCADA software.

SCADA and Telemetry

The controls systems shall include manual, automatic and telemetry capabilities for the pumps and auxiliary systems. The engine start/stop controls shall operate locally at each engine, remotely from the control room, and from the central control station. The automation components of all pumping stations and structures that will eventually be operated and maintained by South Florida Water Management District (SFWMD) must conform to SFWMD standards in order to (1) achieve cost efficiency in design, construction, and operation and maintenance, (2) meet safety, reliability, and performance requirements during routine and emergency operations. The automation components are broadly defined to include hardware, software, communications, and user interface elements.

A.5.5.4 Gated Spillways and Culverts

Gate Operators

Gate operators will be designed based on the size, weight, and hydraulic loading on the gates. The operators will either be electric motor driven through a drum and pulley system or via as an actuator on a stem screw.

Electrical Service

A control center will house a main breaker, combination starter for the gate motor, lighting panel, relay compartment, and a circuit for exterior lighting. Surge suppression will be provided for each electrical/electronic system within or outside the structure.

Control and Monitoring

Duplicate open-close push button station in the control house and at the spillway or culvert structure will be provided for manual gate control. Necessary open, close, automatic control relays, and limit switches will be incorporated in the gate control circuit. Power and control circuits for water level recorders and gate position recorders will be provided.

A.5.5.5 Weir

Water Level Indication

Stilling wells with water level monitoring equipment will be provided on both sides of the weir. Power for the monitoring equipment will be provided by either commercial power or by solar power, depending on the final location of the weir.

A.5.5.6 Telemetry

Each spillway or culvert site that requires remote automation will be equipped with an RTU compatible with the existing SFWMD telemetry system. RTU software will be in accordance with the SFWMD standard load set. The construction plans will contain plans for a fully functioning telemetry system capable of connecting to and communicating with the SFWMD existing system. Additional coordination during the development of plans and specifications will finalize the telemetry requirements.

A.6 SOUTH OF THE REDLINE – DIVERSION & CONVEYANCE

A.6.1 CIVIL - SITE DESIGN

Features identified in the Recommended Plan have been designed to the level of detail necessary to provide cost estimates. Best professional judgment as well as previous project design knowledge for DECOMP were used during plan formulation alternative development and design efforts. Components south of the redline have been identified and sized appropriately according to available data, historic information, and best engineering judgment. All project components will be optimized during PED phase for cost efficiency and performance, incorporating updated data and information as it becomes available.

A.6.1.1 General Status of Completed and Non-Executed Efforts

The following civil site project efforts remain either incomplete or were not initiated:
evaluation of alignments,
site grading,
aesthetics,
relocation of facilities,
required improvements on lands to enable proper construction of components and disposal of material,
requirements of lands for construction, operation and maintenance of the project,
identification of methods for accomplishing relocations to include appropriate lands,
site selection and project development, and
design with respect to recent Levee Safety criteria.

These analyses will be completed in PED.

A.6.1.2 Surveying Mapping Geospatial data

Historical hydrographic and topographic surveys exist for the project area. All survey data collected was performed using conventional means and methods. The existing surveys are 02-019, 02-037, 02-047, 02-142, 07-058, 08-195 and 11-106. Datum's utilized for data collection is as follows: Horizontal coordinates are referenced to the State Plane Coordinate System North Atlantic Datum (NAD) 83 (2007), Florida East Zone (0901). Elevations are in US Survey Feet and referenced to North Atlantic Vertical Datum (NAVD) 88 vertical datum. See Appendix A, Annex C-1 for data points.

A.6.1.3 Access

Access to this project area is primarily from US 27 along the existing L-5 northern access road westward to existing S-8, L-4 and Miami Canal. Access to L-6 is from US 27 along the existing S-7 complex and L-6 areas.

Due to the remote nature of the Miami Canal, site access limitations will be a significant consideration for the CEPP project construction. The limited engineering design information that was incorporated into the development of the final array planning-level cost estimates is documented in the main PIR document Appendix B Cost Engineering.

A.6.1.4 Material Balance and Disposal

Cut and fill quantities will be completed during PED phase to balance the design as much as possible. Material from the construction of canal, from Miami Canal to L-4 and the L-5 conveyance improvements, not suitable to fill in the Miami Canal will be hauled to a certified land fill. Material from onsite earthwork is the source for backfilling the Miami Canal and creation of tree islands mounds.

A.6.1.5 Utility Relocations

Utility impacts, including potential relocations, will need further assessment during the project design phase. Utilities will also have to be provided for S-620, S-621, S-622, S8A, S8W, and S-630. The type and length of utilities will be determined during PED.

A.6.2 GEOTECHNICAL DESIGN

Widening and Deepening of L-5 from S-8 to S-622

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The preliminary design parameters are presented in **Table A-20**.

TABLE A-20. PRELIMINARY SHEAR STRENGTH AND HYDROGEOLOGIC PARAMETERS FOR SOIL AND ROCK AT CEPP

Material	Saturated Unit Weight, pcf	Drained Friction Angle, degrees	Rock Friction Angle, degrees	Rock Cohesion, psf	Horizontal Hydraulic Conductivity, ft/sec	Vertical Hydraulic Conductivity, ft/sec
Peat/Organic Silt	70	25	-	-	1.7E-5	5.7E-6
Caprock	158	-	53	2930	7.3E-4	2.4E-4
Weathered Limestone	131	-	23	1100	2.0E-2	6.7E-3
Deep Sands	124	35	-	-	3.0E-5	1.0E-5
Sand Fill	126	36	-	-	3.4E-6	8.3E-7
Pervious Fill	126	30	-	-	6.6E-4	6.6E-4

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater investigations have been performed in the vicinity of this feature to date, nor are any anticipated during future design.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. South Florida is considered to be one of the most seismically stable locations in the United States (Petersen, Mark D. et. al, 2008). Historically, only minor shocks have occurred, with only one that resulted in minor damage. Additional shocks of suspect origin have been recorded that were felt in the Everglades area. The three Florida shocks of doubtful seismic origin rumbled through the Everglades, La Belle/Fort Myers area in July 1930, Tampa in December 1940, and the Miami/Everglades/Fort Myers area in January 1942. Most authorities attributed these incidents to blasting, but a few contend that they were seismic.

Uniform Building Code Seismic Zone Map (The Disaster Center, 1991) indicated that the entire State of Florida is in Seismic Zone 0 (areas with least potential for seismic activity). Since no capable faults or recent earthquake epicenters are known to exist near the project site and there is no dam impoundment included in this project feature, per Design Criteria Memorandum (DCM) No. 6, evaluation of liquefaction is not required.

f. Preliminary foundation design and slope stability analyses. There is no foundation to design for with this feature. The slopes shall be cut to a shallower or equal angle than currently that of the original design cut slope. Typical original design cut slopes in soils were 1V:2H and 1V:1H in rock.

g. Excavatability analysis. Rock rippability will be evaluated based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the canal can be widened and deepened by standard long arm hydraulic excavator, dredge or by dragline excavation equipment within the layers of peat and organic materials in the top 4-6 feet from the bank. Below this level excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or underwater blasting may be required to remove unrippable rock strata. Excavated cobbles and grains less than 3 inches in effective diameter can be loaded onto dump trucks by rubber-tired loaders and hauled off to the Miami Canal for canal filling. Larger cobbles and boulders can be crushed and mixed with the minus 3 inches of soil and rock.

i. Potential borrow and disposal sites. The canal is the borrow area for inorganic material destined to be delivered to the Miami Canal filling area. The organics are to be disposed to a designated area to be determined during the design phase.

j. Seepage and groundwater control. There is a possibility that widening and deepening of the canal may induce higher toe hydraulic gradients depending on the design headwater and tailwater conditions. A seepage analysis may be conducted during the design phase to verify the adequacy against internal erosion of the underlying silts.

S-620 and S-621 Gated spillways on L-5

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater investigations have been performed in the vicinity of this feature to date, nor are any anticipated during future design.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. It is anticipated that, the spillway structure foundations will be founded on underlying limestone or compacted cohesionless fill. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. A heave or uplift evaluation will be required to design the tremie concrete slabs during the design phase. Slope stability analyses other than temporary cut slope evaluation during the design phase will not be required.

g. Excavatability analysis. Rock rippability has been estimated at this time and these estimates will be evaluated further based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the spillway structure areas can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank and through the existing embankments. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Backfill will be accomplished with compacted layers of granular backfill with rewatering. Excess excavated inorganic cobbles and grains less than 3 inches in effective diameter can be loaded onto dump trucks by rubber-tired loaders and hauled off to the Miami Canal for canal filling. Larger cobbles and boulders can be crushed and mixed with the minus 3 inches of soil and rock.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as backfill for the structures. Excess inorganic material will be destined to be delivered to the Miami Canal filling area. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. Sheetpile cofferdams with tremie concrete slabs to facilitate dewatering and dry construction are typically incorporated into the construction of these features. Discharge will be to the canal after appropriate treatment. Dewatering is typically accomplished by sump pumps within the excavation pit with supplemental groundwater lowering via well point rows. Other methods for dewatering may be utilized for construction efficiency and cost savings.

S-620 gated spillway on L-6

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater investigations have been performed in the vicinity of this feature to date, nor are any anticipated during future design.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. It is anticipated that, the spillway structure foundations will be founded on underlying limestone or compacted cohesionless fill. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. A heave or uplift evaluation will be required to design the tremie concrete slabs during the design phase. Slope stability analyses other than temporary cut slope evaluation during the design phase will not be required.

g. Excavatability analysis. Rock rippability has been estimated at this time and these estimates will be evaluated further based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this

h. Anticipated construction techniques. It is anticipated that the spillway structure areas can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank and through the existing embankments. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Backfill will be accomplished with compacted layers of granular backfill with rewatering. Excess excavated inorganic cobbles and grains less than 3 inches in effective diameter can be loaded onto dump trucks by and hauled off to the Miami Canal for canal filling. Larger cobbles and boulders can be crushed and mixed with the minus 3 inches of soil and rock.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as backfill for the structures. Excess inorganic material will be used at the Miami Canal filling area. The organic materials will then be disposed in an area to be determined during the design phase.

Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. Sheetpile cofferdams with tremie concrete slabs to facilitate dewatering and dry construction are typically incorporated into the construction of these features. Discharge of dewatering effluent will be to the canal after appropriate treatment. Dewatering is typically accomplished by sump pumps within the excavation pit with supplemental groundwater lowering via well point rows. Other methods for dewatering may be utilized for construction efficiency and cost savings.

<u>Miami Canal new spillway S-623</u>

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date. However, a dewatering evaluation will be performed with seepage analysis during the design phase.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. It is anticipated that, the spillway structure foundations will be founded on underlying limestone or compacted cohesionless fill. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. A heave or uplift evaluation will be required to design the tremie concrete slabs during the design phase. Slope stability analyses other than temporary cut slope evaluation during the design phase will not be required.

g. Excavatability analysis. Rock rippability has been estimated at this time and these estimates will be evaluated further based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the spillway structure areas can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank and through the existing embankments. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Backfill will be accomplished with compacted layers of granular backfill with rewatering. Excess excavated inorganic cobbles and grains less than 3 inches in effective diameter can be loaded onto dump trucks and hauled off to the Miami Canal for canal filling. Larger cobbles and boulders can be crushed and mixed with the minus 3 inches of soil and rock.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as backfill for the structures. Excess inorganic material will be delivered to and used at the Miami Canal filling area. The organics will be disposed of in an area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. Sheetpile cofferdams with tremie concrete slabs to facilitate dewatering and dry construction are typically incorporated into the construction of these features. Discharge of dewatering effluent will be to the canal after appropriate treatment. Dewatering is typically accomplished by sump pumps within the excavation pit with supplemental groundwater lowering via well point rows. Other methods for dewatering may be utilized for construction efficiency and cost savings.

<u>Deepen S-8 for extra capacity</u>
a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date. However, a dewatering evaluation will be performed with seepage analysis during the design phase.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. It is anticipated that, the pump structure foundation will be founded on underlying limestone. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. A heave or uplift evaluation will be required to design the tremie concrete slabs during the design phase. Slope stability analyses other than temporary cut slope evaluation during the design phase will not be required.

g. Excavatability analysis. Rock rippability has been estimated at this time and these estimates will be evaluated further based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the pump structure area can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank and through the existing embankments. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Backfill will be accomplished with compacted layers of granular backfill with rewatering. Excess excavated inorganic cobbles and grains less than 3 inches in effective diameter can be loaded onto dump trucks and hauled off to the Miami Canal for canal filling. Larger cobbles and boulders can be crushed and mixed with the minus 3 inches of soil and rock.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as backfill for the structures. Excess inorganic material will be destined to be delivered to the Miami Canal filling area. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. A sheetpile cofferdam with tremie concrete slab to facilitate dewatering and dry construction is typically incorporated into the construction of these types of features. Discharge of dewatering effluent will be to the canal after appropriate treatment. Dewatering is typically accomplished by sump pumps within the excavation pit with supplemental groundwater lowering via well point rows. Other methods for dewatering may be utilized for construction efficiency and cost savings.

L-4 Degrade

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. There is no foundation to design for with this feature. The slopes shall be cut to a shallower or equal angle than currently that of the original design levee side slopes of 1V:3H. A riprap blanket with bedding may be needed on each cut face depending on design flow velocities through the gap which will be determined during the design phase.

g. Excavatability analysis. Excavation can be conducted by standard excavating equipment of dozers, loaders, and dump trucks, as this is a degrade of an existing levee. Some excavation may be under wet conditions. No rippability evaluation is anticipated.

h. Anticipated construction techniques. Levee material will be removed by excavators and hauled off to the Miami Canal as fill. Cobbles and boulders greater than 6 inches nominal diameter from the levee may need to be processed before placement in the Miami Canal. Riprap and bedding may be required to be placed on the end cuts of the breach for erosion protection.

i. Potential borrow and disposal sites. Excavated inorganic material will be destined to be delivered to the Miami Canal filling area. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course. Riprap and bedding will be imported from offsite sources.

j. Seepage and groundwater control. There may be excavation in a wet condition and placement of riprap in the wet or dewatered condition. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration.

A.6.2.1 General Status of Completed and Non-Executed Efforts

Despite the geological information presented here, some data gaps do exist which require further investigation. Portions of the Miami Canal from S-339 to S-8 need more exploratory borings to verify the thickness of the peat and top of limestone for the various alternatives. The south side of L-4 would also require further exploratory borings and perhaps hydraulic testing.

A.6.2.2 Soils

Three major soil types are found in the Everglades: 1) peat soils, which are high in organic content and are comprised of partially decayed plant material (two types of peat can be found in the Everglades: Everglades and Loxahatchee peat types), 2) marl soils, which have lower organic content and are comprised of calcitic mud deposited from calcareous periphyton (Gleason and Spackman 1974) and 3), tree island soils that have greater portions of mineral components than peat soils, but are very similar in that their origins are both plant material. The origin and development of peat and marl soils are greatly dependent upon water depth and resulting wetland vegetative communities. Alteration of the hydroperiod and diminished surface water inundation may also alter the vegetation communities and subsequent changes in soil type and depth or elevation could occur.

A.6.2.3 Geology

South of central Broward County to the Blue Line, the composition of the Fort Thompson Formation mixed clastic carbonates (poorly consolidated marine limestone, quartz sandstone, and sandy limestone) changes to predominately marine limestone that were deposited in marine platform margin and open marine tropical conditions similar to those observed in the present-day southern Florida Keys. The oolitic Miami Limestone often outcrops at the surface near the Blue Line and forms approximately 10 to 15 ft. of caprock overlying the Fort Thompson Formation. At the Blue Line, the Fort Thompson Formation is a karstic limestone in southern Broward/Miami-Dade Counties and has been characterized by Cunningham et. al (2006) into 16 distinct lithofacies representing freshwater, platform margin, ramp, and open marine carbonate depositional environments. Subsequent dissolution of these limestones during low sea levels resulted in the development of karst with extensive vugs and conduits throughout the vertical sequence of rock. Thus, the gradation of lithologies from mixed clastic-carbonates near the Red Line to karstic marine carbonates at the Blue Line affects the porosity and permeability of the sedimentary package.

Geologic information gathered from the pre-widening area of L-5 in 2000 by Nodarse and Associates (Figures ANNEX-3 and ANNEX-4), the subsurface characterization of L-5/L-4 for the Decompartmentalization and Hydrologic Sheetflow study (DECOMP) (U.S. Army Corps of Engineers, 2011) and the Wolff WPC, 2009 study of the Miami Canal south of S-339 yielded the following results from top to bottom (ANNEX G-2):

*__Layer I: Fill__. This layer consists of localized areas of fill adjacent to existing canals at the time of the construction of these canals. The material is predominately sandy fill with some gravel, trace clay, some gravel and some shell in L-4 and L-5. By the Miami Canal (L-23) in Broward County, the fill is predominately limestone (crushed rock) fill. Both fill material varies in thickness from 0.5 foot to 6 feet. The fill has standard penetration N-values between 8 and 84 blows per foot depending upon the degree of material compaction. If groundwater is present in the fill, it occurs between 2 and 4.5 feet below grade.

***Layer 2: Interbeds of Organic Sand and Clay Including "Peaty" Clay.** This layer consists of alternating beds of organic sand and clay. Sand unit is predominately well-graded (poorly sorted) with some shell fragments and trace clay. Thickness ranges from 0.1 to 9 feet. In the L-5/L-4 area, standard penetration N-values varies between 2 and 68 blows per foot. The clay unit has trace gravel, sand and some shell fragments. In some places, the clay unit is carbonaceous or "peaty" (fibrous) and in some places, "fat" clay is present. Thickness ranges from 0.2 to 4.5 feet for the clay unit. Standard penetration N-values range from 0 to 11 blows per foot in the northern hydropattern restoration area. In Broward County, this clay unit appears to be laterally continuous. If groundwater is present, it occurs between 6.6 and 8.5 feet below grade within these units.

***Layer 3: Limestone**. Underlying the unconsolidated material of fill/organic sand and clay is limestone that is fossiliferous, vuggy, moderately to intensely weathered that is also slightly to highly fractured. Clay infilling of the voids is apparent in some areas. No unconfined compressive strength tests were conducted. Rock quality designations ranged from 0 to 77 percent with an average value of 25 percent. In some places, the limestone is interbedded between the organic sand/clay units.

A.6.2.4 HTRW

The Corps will review the HTRW condition of the affected parcels and ensure that the proper due diligence is performed in accordance with ER 1165-2-132 prior to certifying lands for construction. Should remediation of HTRW contamination be required, it is the responsibility of the SFWMD, the non-Federal, sponsor and is not a creditable cost to the project.

A.6.3 HYDRAULIC DESIGN

A.6.3.1 General Status of Completed and Non-Executed Efforts

Features identified in the Recommended Plan have been designed to the level of detail necessary to provide cost estimates and determine feasibility of hydraulic design. All components at the redline have been identified, sized appropriately according to available modeling data, historic information, and best engineering judgment. All project components will be optimized during PED phase for cost efficiency and performance, incorporating updated data and information as it becomes available. General hydraulic design of all identified components south of the redline are described in the following sections.

A.6.3.2 Hydraulic Design – General

This section provides a brief overview of the hydraulic design criteria, parameters, intent/purpose of project features. Detailed hydraulic design of individual components is described in later sections, including hydraulic design data sheets. Detailed data resulting from model simulations may be found separately in Appendix A, Annex A-1. Currently, all elevations are referenced to NGVD 29; elevations will be provided in both NGVD 29 and NAVD 88 when revised during PED.

A.6.3.2.1 Design Criteria and Parameters

A.6.3.2.1.1 Canals

Canal side slopes are generally steeper than is found in many sandy regions. This is due to the limestone geology allowing for near vertical slopes in some locations. Generally, there is a preference for some slope in case of sand lenses and well weathered limestone that with time deteriorates to gravel and sand sizes, therefore most canal side slopes are 1V:2H.

A.6.3.2.1.1.1 Manning's Roughness Coefficient Determination

A Manning's roughness coefficient, or n value, of 0.035 was used for canal design in conveyance of design flows. The Palm Beach County region where the CEPP components and canals will be constructed has geological features characteristically described as limestone covered with a relatively thin layer of overburden (peat, marl, muck). For cost purposes, it is assumed that blasting will be the method of excavation for canals; however, the final method (blasting, ripping, etc.) will be determined in PED. These types of excavation methods leave a relatively coarse bottom and bank with sharp edged rocks and rubble. With canal age or maturation, some of these irregularities will become less defined resulting in a smoother perimeter with lower roughness values. Aquatic growth is expected along the upper banks where the overburden lies, but the design depths should be relatively free of plant roots extending from bottom to surface obstructing flow. Floating plants will be controlled by spray and harvest methods. The Manning's n value used was obtained from investigating various sources and noting them as follows.

In C&SF Project General Studies and Reports, Part I, Supplement 18, the following was noted; a value of at least 0.035 should be used where channels are constructed primarily in rock. This value is for channels with no appreciable erosion and with rapid-growth vegetation along the upper banks because of the organic soil overburden. Other sources provide Manning's n values within the same ranges as SCS and USGS for similar type canals. Brater and King's Handbook of Hydraulics, 7th ed., provides an n value of 0.035 for canals with rough stony beds and weeds on earth banks in fair condition. From the preceding investigations, and experience at the Jacksonville District in Florida, a Manning's n value of 0.035 appears to be appropriately applied to the design to satisfy criteria outlined by all referenced sources as the minimum acceptable value.

In detail design phase, the canal parameters may be modified for optimal benefit cost ratio.

A.6.3.2.1.2 Head Loss

Due to the relatively flat topography throughout the project area, the hydraulic head losses across many of the control structures are low, resulting in the design of larger structures (number and size of barrels, bays, etc.) than may typically be assumed for other regions. The use of pumps was avoided wherever possible to reduce operation and perpetual maintenance costs. During PED phase, SAJ expects to optimize system operations and therefore structure sizes for cost and performance efficiencies.

A.6.3.2.1.3 Flow and Velocity

Design flow rates for all water control structures were determined based on RSM-BN model outputs and existing canal and structure capacities. To capture cost impact adequately, structures and canals were designed for maximum capacity scenarios. Optimization of these features will be conducted during the PED phase for performance and cost efficiency.

Canals were designed to maintain a velocity of 2.0 fps or less to avoid potential erosion damage. Given the small topographical relief of the project area, this is typically the condition under normal operations, regardless.

A.6.3.2.1.4 Water Control Structures

The CEPP proposed plan will have multiple water control structures (S-620, S-621, S-622, and S-630). The function of the control structures are to convey deliveries from the L-6 Canal westward through the L-5 Borrow Canal to the S-8 pump station. The conveyance structure S-620 will be gated culverts replacing the existing plug at the downstream extent of the L-6 Canal, just upstream of the S-7 pump station. S-620 is sized to a capacity of 500 cfs, matching the existing L-6 canal conveyance capacity. The S-621 gated spillway is located in the STA 3/4 Outflow Canal, and will be used to block flows from the STA 3/4 from entering the L-5 Canal when L-6 deliveries are being made. The S-621 structure was sized to 2,500 cfs, matching the capacity of the S-7 pump station. The S-622 gated spillway will replace the existing plug in the L-5 Canal, located near the rock pits at the southwest corner of the STA 3/4. The spillway was sized to match the L-6 deliveries quantity of 500 cfs. The S-630 pump station is located along the L-4 Canal, west of the S-8 pump station, to provide water supply deliveries to the Seminole Tribe of Florida's Big Cypress Reservation via the G-409 pump station and water supply deliveries to STA-5 and STA-6, as well as to act as a divide structure to stage up water surface elevations in the L-4 Canal to flow over the proposed degrade of the L-4 Levee. The pump is sized to 360 cfs to provide concurrent water supply deliveries to the G-409 pump station (190 cfs) and water supply deliveries to STAs 5 and 6.

A.6.3.2.1.4.1 Gated Culverts

The entire CEPP project includes numerous gated box culverts across the entire project area. Construction material for all culverts is to be cast in place concrete.

An entrance loss coefficient value of 0.9 (assumed due to gate-added turbulence around inlet) and exit loss coefficient of 1.0 was used for all gated culvert structures. Also, the Manning's friction or energy loss coefficient was assigned 0.013 for all culverts. All major conveyance culverts were designed to remain submerged year round to reduce aquatic growth within, thereby better maintaining design friction head losses. All gated culvert sites were designed with a minimum of two culverts to allow maintenance activities to coincide, however with reduced capacity operations.

A.6.3.2.1.4.2 Gated Spillways

The CEPP Recommended Plan includes the design of two ogee weir concrete spillways with steel vertical lift gates located in line with the STA 3/4 Outflow Canal and in the L -5 Canal to replace the existing plug. The spillways were designed to be in conformity of engineering guidance found in USACE EM-1110-2-1603. All ogee spillways have vertical gates for controlled discharge operations. The spillways were designed with minimal head differential for conveyance energy. The S-621 spillway was designed with a 0.2 foot head differential. This design constraint is based on the assumption that the combination of L-5 Canal conveyance improvements and proposed S-8 pump station replacement/improvements will provide the ability to move the current maximum capacity of the STA 3/4 Outflow Canal. The S-622 spillway was designed with a 0.1 foot head differential. The structure's intent is to replace the plug existing in the L-5 Canal, and to have a little head differential as possible. The spillway is designed for 500 cfs to match the incoming deliveries from the L-6 Canal. SAJ acknowledges this low head differential constraint and accepts optimization will be required, including Value Engineering appropriate structure type for this function.

A.6.3.2.1.4.3 Pump Stations

The CEPP Recommended Plan proposes to construct a new pump station, S-630, for conveyance to the Seminole Tribe of Florida's Big Cypress Reservation and to provide flow over the degraded L-4 Levee. Additionally, the S-7 and S-8 pump stations will be utilized to provide L-6 and L-5 conveyances.

A.6.3.3 L-6 Diversion and Conveyance

A.6.3.3.1 General Information

The CEPP features at the redline include new structures and canal modifications to convey flows from the L-6 Canal westward to the modified S-8 pump station to distribute along L-4. A portion of the L-4 southern levee will be degraded to create an outlet for flow distribution into WCA-3A. All south of the redline components are within Palm Beach County, north of Water Conservation Area (WCA-3A).

A.6.3.3.1.1 Purpose

The purpose of the L-6 diversion and conveyance component of the CEPP project is to be able to move additional outflows from STA 3/4 and STA 2 from the L-6 to provide hydropattern restoration to the northwest WCA 3A. By making the proposed improvements to the L-5 Canal, available flow from either L-6 Canal or STA 3/4, or both, can be moved westward to the L-4 Canal, where the proposed levee degrade will provide flows to northwest WCA 3A.

A.6.3.3.1.2 Location

The L-6 diversion and conveyance component begins along the L-6 Canal, near the eastern intersection of STA 2 and Compartment B. The component spatial extent includes the L-5 Canal and the L-4 Canal. The component boundaries are located on the Palm Beach/Broward County line.

A.6.3.3.1.3 Features

The CEPP project has the following L-6 diversion features south of the redline:

Structures:

- S-620 Gated Culvert (CS-1)
- S-621 Gated Spillway (CS-2)
- S-622 Gated Spillway (CS-3)
- S-630 Pump Station

Canals:

- L-5 Canal

Figure A-4 and **Figure A-5** illustrate all feature locations for South of the Redline (structures and canals are not to scale or geographically referenced). Detailed design analysis for hydraulic components south of the redline pertaining to L-6 diversion and conveyance can be found in supplemental documents located in Appendix A, Annex A-1.

FIGURE A-4. NORTH OF REDLINE/AT REDLINE LOCATION MAP

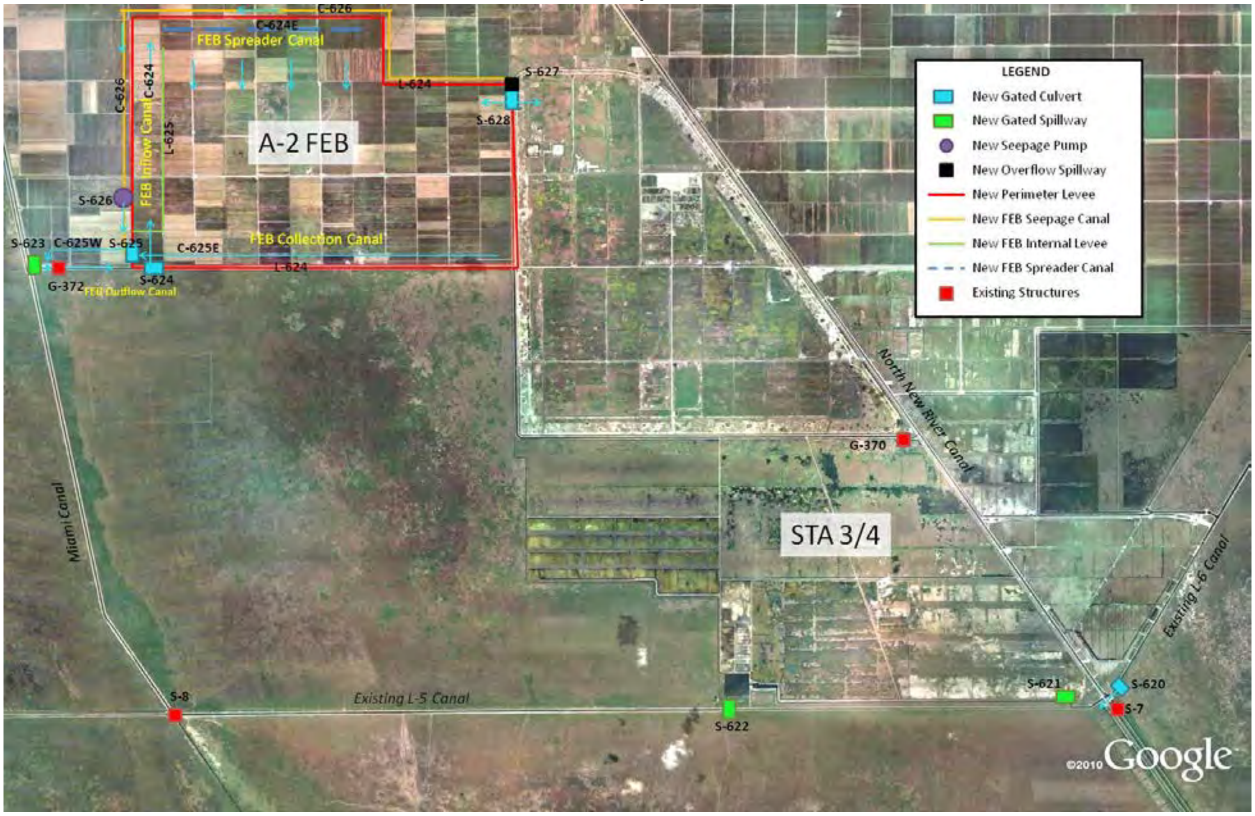


FIGURE A-5. L-4 LOCATION MAP

A.6.3.3.2 Hydraulic Design

A.6.3.3.2.1 Proposed Water Control Structures

A.6.3.3.2.1.1 Gated Culverts

S-620 Gated Culvert (CS-1)
The structure is an outlet control structure to allow conveyance from the L-6 Canal to the eastern (remnant) L-5 Canal, replacing the existing plug at the most southern end of the L-6 Canal. S-620 is a two-barreled gated box culvert structure. The design flow is 500 cfs with a design hydraulic head of 0.5 ft. The structure is a typical box culvert with dimensions of 8 ft by 8 ft with vertical slide gates and a total length of 75 ft. The upstream and downstream inverts are set at elevation -3.5 ft NGVD. The design velocity through the structure is 4.0 fps.

A.6.3.3.2.1.2 Gated Spillways

S-621 Gated Spillway (CS-2)
S-621 is a gated spillway that will serve as a divide structure to separate STA 3/4 outflows from the eastern (remnant) L-5 Canal when L-6 deliveries are being made to the L-5 Canal. When open, S-621 will allow for a portion of the STA 3/4 outflow discharges to be delivered to the S-7 pump station. During normal operations, including L-6 diversion flows, CEPP will direct the majority of STA 3/4 discharges westward to the modified S-8 pump station, and operation of S-7 pump station to deliver STA 3/4 discharges to WCA-2A is anticipated primarily during peak discharge events. S-621 is a three-bay gated spillway. The design flow is 2,500 cfs with a design hydraulic head of 0.2 feet. The design flow was set to match that of the S-7 pump station. The spillway consists of three gates with dimensions of 23 ft wide by 12.5 ft high. The crest elevation is set to 1.0 ft NGVD. The upstream and downstream aprons are set at an elevation of -5.0 ft NGVD, with apron lengths of 30 ft. S-621 is located in line with the STA 3/4 Outflow Canal, just north of the L-5 Canal. During PED, the design requirements for S-621 will be revisited, and this structure may be removed from the CEPP project.

S-622 Gated Spillway (CS-3)
S-622 is a gated spillway that will replace the existing plug in the L-5 Canal to hydraulically connect the eastern and western portions of the canal. S-622 is a three-bay gated spillway. The design flow is 500 cfs with a design hydraulic head of 0.1 feet. The spillway consists of three gates with dimensions of 15 ft wide by 10 ft high. The crest elevation is set to 5.00 ft NGVD. The approach apron and discharge apron inverts are set at an elevation of 0.00 ft NGVD with lengths of 33 ft. S-622 is located in line with the L-5 Canal, just south of the former Griffin rock pits near the southwest corner of STA 3/4.

A.6.3.3.2.1.3 Canals

L-5 Canal Improvements
In order to accommodate the proposed flows through the L-5 Canal, conveyance improvements must be made to both the eastern (remnant canal) and western portions. An existing plug located about midway along the canal (south of the former Griffin rock pits) will be removed and replaced with gated spillway S-622 in order to divide flows for varying conveyance scenarios. The CEPP modifications to the eastern remnant L-5 Canal will accommodate 500 cfs, and the CEPP modifications to the west L-5 Canal will accommodate 3,000 cfs. The design HW and TW for the improved canal were 12.00 ft NGVD and 10.00 ft NGVD, respectively.

TABLE A-21. L-5 REMNANT CANAL IMPROVEMENTS

Length	Side Slope L/R	Canal Bottom Width	Canal Bottom Elevation	Average Depth
Feet	V:H	Feet	Ft, NGVD	Feet
31,000	1:1.5	50	-5.1	14.6

TABLE A-22. L-5 WESTERN CANAL IMPROVEMENTS

Length	Side Slope L/R	Canal Bottom Width	Canal Bottom Elevation	Average Depth
Feet	V:H	Feet	Ft, NGVD	Feet
45,000	1:1.5	100	-5.6	16.0

TABLE A-23. L-5 CANAL IMPROVEMENTS COMPARISON

	Station	Existing Canal at 1,550 cfs				Improved Canal at 1,550 cfs		
		WSE	Channel Invert	Mean Channel Velocity	Flow Area	WSE	Channel Invert	Mean Channel Velocity
		Ft, NGVD	Ft, NGVD	fps	Sq ft	Ft, NGVD	Ft, NGVD	fps
Eastern Canal	77189.11	14.49	-2.94	0.27	1828.74	11.96	-5.1	0.31
	71197.72	14.46	-2.1	0.60	1075.69	11.94	-5.1	0.39
	65198.22	14.4	-0.8	0.61	1036.59	11.93	-5.1	0.39
	59196.34	14.34	-0.29	0.71	934.25	11.91	-5.1	0.39
	52199.00	14.26	0.4	0.66	965.65	11.89	-5.1	0.39
	47693.76	14.2	0.13	0.65	923.57	11.87	-5.1	0.39
Western Canal	44696.79	14.06	-4.2	1.63	1929.01	11.82	-5.6	1.36
	39196.49	13.82	-5.0	1.61	1933.16	11.63	-5.6	1.38
	30697.78	13.25	-0.6	1.67	1861.23	11.34	-5.6	1.40
	21692.23	12.45	-4.9	1.85	1657.15	11.01	-5.6	1.45
	12690.65	11.55	-6.69	2.11	1460.62	10.46	-5.6	1.48
	691.9784	10.11	-5.1	1.99	1511.01	10.02	-5.6	1.56

A.6.3.3.2.1.4 Pump Stations

S-630 Pump Station

The S-630 structure is a small 360 cfs pump station on the L-4 Canal, used to maintain existing water supply deliveries to the Seminole Tribe of Florida's Big Cypress Reservation, STA-5, and STA-6, and to stage up water in the L-4 Canal to allow discharge over the L-4 Levee degrade. Currently, water supply deliveries can be made from the G-404 pump station (at the eastern terminus of L-4) through the existing gap in the south L-4 Levee. The seepage pump will be equipped with four 90 cfs electric motor driven pumps and a liquefied petroleum gas (LPG) generator, which will serve as an alternative power source in case of power outages.

Pump Rates

The S-630 pump station will be located west of the L-4 levee degrade, with the intent to provide water supply deliveries to meet the demands of the G-409 pump station (190 cfs) and to provide water supply to STA-5/6. The S-630 pump station will also be used to provide lift to allow water to overflow the L-4 southern levee degrade. The pumping rate of 360 cfs was established to provide concurrent water supply deliveries to the existing G-409 pump station (190 cfs) and to provide water supply demands to STA-5 and STA-6.

Pump Mix

Pump mixes were based on a minimum of two bay pump stations to minimize risk of impact to private lands should a single pump fail during critical times. All small pump stations will be equipped with electric motor driven pumps that have diesel generators or pumps for an alternative power source in cases of power outages. One criterion for all pump mixes was to utilize duplicate pump sizes as much as possible to reduce operation and maintenance costs. This is accounted for through a reduction in different spare parts required and focusing mechanical expertise. Another criterion was to provide a pump mix that allows a smooth pump rate change interval from start-up to full capacity. The S-630 pump station will be equipped with four 90 cfs electric motor driven pumps for normal operations and an LPG generator to provide backup power.

Pump Stages

Pump stages were defined by the following pumping parameters:

Intake Water Surface Elevations:

Maximum Non-Pumping: Highest canal or pool stage that can be expected to occur.
Maximum Pumping: Maximum canal or pool stage that can be pumped with any increase in stage requiring the pump to be turned off. In most cases, Maximum Non-Pumping and Maximum Pumping stages are identical.
Start Pumping: Canal or pool stage when pump may be turned on as defined by system conditions, typically on the increasing limb.
Normal Drawdown: Expected local drawdown at the pump station intake.
Minimum Drawdown: Lowest local drawdown stage before pump is required to be turned off.
Minimum Non-Pumping: Lowest canal or pool stage that can be expected to occur under non-pumping conditions.

Discharge Water Surface Elevations:

Maximum Non-Pumping: Highest canal or pool stage that can be expected to occur.
Maximum Pumping: Maximum canal or pool stage that can be pumped to, pump is subsequently turned off until stage decreases.
Normal Pumping: Expected normal pool elevations for impoundments and design tailwater stages for conveyance canal pump stations (flood damage reduction drainage discharge).
Minimum Pumping: Lowest canal or pool stage expected when pump may be turned on.
Minimum Non-Pumping: Lowest canal or pool stage that can be expected to occur under non-pumping conditions. In most cases, Minimum Pumping and Minimum Non-Pumping elevations are identical.

A.6.3.3.2.2 Existing Structures

S-7 Pump Station
S-7 is an existing pump station that is currently used to discharge runoff water via the North New River Canal, as well as provide an outlet for STA 3/4 discharges, into WCA-2A. The pump station is equipped with three 830 cfs diesel pumps for a total capacity of 2,490 cfs. For CEPP, the majority of STA 3/4 discharges will be delivered to the modified S-8 Pump Station for delivery to WCA 3A, and operation of the S-7 Pump Station will likely be limited to peak events. The pump station is located in the alignment of the North New River Canal at the northwestern corner of WCA-2A.

S-8 Pump Station
S-8 is an existing pump station that is currently used to discharge runoff water via the Miami Canal, as well as provide an outlet for STA 3/4 discharges, into WCA-3A. CEPP will maintain this existing design capacity for the S-8 complex through a combination of the following design considerations: pump station design modifications, a new hydraulic connection from S-8 to the degraded L-4 Levee (New S-8A), utilization of the existing G-404 pump station (570 cfs design capacity), and leaving the 1-2 mile segment of the Miami Canal as available getaway conveyance capacity during peak flow events. For CEPP, the S-8 pump station and/or G-404 may require design modifications (or possible replacement). The Recommended Plan cost estimate includes costs for the potential S-8 complex modifications, which are included as the new S-8A (canal connection to L-4 and two culverts structures). During PED, the following design uncertainties will be assessed/reassessed in further detail: modifications to S-8 and/or G-404, to address pump efficiency concerns; the proposed S-8A culvert and associated canal connecting the Miami Canal to the L-4 Canal; and the required length of the unmodified Miami Canal to maintain hydraulic getaway conveyance capacity. Flood control operation capability will be maintained during S-8 modification construction. S-8 is equipped with four 1,040 cfs diesel pumps for a total capacity of 4,160 cfs. The pump station is located in the alignment of the Miami Canal at the northern boundary of WCA-3A.

A.6.3.3.2.3 Existing Canals

L-4 Canal
The L-4 Canal is located west of the S-8 pump station and currently conveys water from the Miami Canal and the L-5 Canal to the G-409 pump station where it is provides water supply to the Seminole Tribe of Florida's Big Cypress Reservation, to STA 5/6, or to Northwest WCA-3A via the L-4 gap and the L-3 Extension Canal.

L-6 Canal
The existing L-6 Canal currently conveys water from the STA-2 via outflow structure G-335. Currently available information suggests the in-bank capacity of the canal can adequately convey the 500 cfs required for L-6 delivers to the L-5 Canal. If the existing conditions do not meet required criteria, operational stage changes or modifications may be required. Further survey analysis will be conducted during PED to verify the existing conditions.

A.6.3.4 Risk and Uncertainty

This section presents qualitatively the risk and uncertainty associated with the project as designed for this PIR. Understanding that the current USACE philosophical approach to Feasibility Studies is to be quick and limit analyses to that for benefit and cost determinations, acknowledging risk and uncertainty in the hydraulic design of the project will be an important part of the risk registry. The overall approach to the hydraulic design was to be conservative enough to capture expected costs without being unrealistic in overestimation, yet not to underestimate beyond what optimization and the savings that could be realized during PED phase efforts.

A.6.3.4.1 Computer Software Tools

Several hydrologic and hydraulic computer software tools were utilized in the formulation of alternatives and the Recommended Plan. Interpretation of hydraulic design results should consider the inherent strengths and limitations of the underlying hydrologic and hydraulic tools. Additional descriptions of the modeling tools are provided in Appendix A, Section A.8.1 (Modeling Strategy).

A.6.3.4.2 Hydraulics and Hydrology Lowering Risk in Design

<u>S-8 Pumping Station Tailwater Impact with Miami Canal Backfill</u>
The Miami Canal affords discharge capacity for the S-8 Pump Station when it is operated for flood protection purposes. The canal provides length necessary for the discharge to flow from clear canal path into the emergent marsh vegetation with relatively high hydraulic head loss. To offset the potential tailwater impact, three additional features are provided: (1) routing or diversion of a portion of the discharge will be made to the west with discharge into WCA-3A with L-4 levee degrade, (2) inclusion of the 360 cfs S-630 Pump Station that pumps water further west out of the western diversion conveyance, and (3) leaving a designed section of non-backfilled length of the Miami Canal from the S-8 Pump Station. With CEPP implementation, there is little risk that flood protection will not be able to be maintained as it functions and is operated today. Also, lowering the risk even further is the construction of the FEB(s) that function as surge tanks within the basin that uptake excess runoff before triggering the S-8 Pump Station, reducing the pumping to large precipitation events or when the FEB is full with no storage availability.

For CEPP, the S-8 pump station and/or G-404 may require design modifications (or possible replacement). The Recommended Plan cost estimate includes costs for the potential S-8 complex modifications, which are included as the new S-8A (canal connection to L-4 and two culverts structures). During PED, the following design uncertainties will be assessed/reassessed in further detail: modifications to S-8 and/or G-404, to address pump efficiency concerns; the proposed S-8A culvert and associated canal connecting the Miami Canal to the L-4 Canal; and the required length of the unmodified Miami Canal to maintain hydraulic getaway conveyance capacity.

<u>Miami Canal Backfill to Sheetflow Characteristics</u>
The Miami Canal is cut nearly perpendicular to topographical contours through WCA-3A. As such, water is “short-circuited” through the wetlands versus historic shallow sheetflow across the floodplain. To investigate how backfilling the canal may impact flow, a 2-dimenisional model using the Adaptive Hydraulics Modeling (AdH) computing software was constructed and simulation of various “plug” or backfill lengths were made with various configurations. It was found that a plug length of simple configuration, e.g. no berm lateral extensions into the marsh, of 4,000 feet caused canal flows to leave the canal, enter the marsh, and continue southerly as sheetflow. Since the design backfill is of longer length, there is little risk that the planned feature will not work as intended.

A.6.3.5 HYDRAULIC DESIGN DATA SHEETS

TABLE A-24. S-620 GATED CULVERT
HYDRAULIC DESIGN DATA SHEET

Location Southern extent of L-6 Canal; x = 807,929 y = 728,365

Purpose S-620 is proposed to be a two-barreled 8 ft by 8 ft gated box culvert to control outflow from the L-6 Canal to the L-5 Canal. The structure will replace the existing plug. S-620 will be located at the southern end of the L-6 Canal, approximately 0.15 miles north of S-7.

Design Conditions	Discharge	500	cfs
	Headwater Elevation	12.50	feet, NGVD 29
	Tailwater Elevation	12.00	feet, NGVD 29
Culvert Data	Number of Barrels	2	
	Barrel Type	Concrete Box Culvert	
	Box Width	8	feet
	Box Height	8	feet
	Culvert Length	75	feet
	Upstream Invert	-3.50	feet, NGVD 29
	Downstream Invert	-3.50	feet, NGVD 29
	Natural Water Table	9.00	feet, NGVD 29
	Headwall - HW Elevation	TBD	feet, NGVD 29
	Headwall - TW Elevation	TBD	feet, NGVD 29
	Wingwall - HW Elevation	TBD	feet, NGVD 29
	Wingwall - TW Elevation	TBD	feet, NGVD 29
Canal Data	Side Slopes (V:H)	1 on 2	
	Upstream Bottom Width	28	feet
	Upstream Bottom Elevation	-4.00	feet, NGVD 29
	Downstream Bottom Width	28	
	Downstream Bottom Elevation	-4.00	feet, NGVD 29
Energy Dissipation	**Riprap Requirements**		
	Rip Rap Design Velocity	4	fps
	Upstream Length	TBD	feet
	Upstream Protection Elevation	TBD	feet, NGVD 29
	Downstream Length	TBD	feet
	Downstream Protection Elevation	TBD	feet, NGVD 29

TABLE A-25. S-621 GATED SPILLWAY
HYDRAULIC DESIGN DATA SHEET

Location Near intersection of STA 3/4 Outflow Canal and L-5; x = 804,442 y = 729,629

Purpose S-621 controls flows from STA 3/4 Outflow Canal into the L-5 Canal

Design Conditions	Discharge	2,500	cfs
	Headwater Elevation	12.20	feet, NGVD 29
	Tailwater Elevation	12.00	feet, NGVD 29
Crest Data	Shape	Ogee	
	Design Head (Hd)	11.2	feet
	Net Crest Length	69.0	feet
	Crest Elevation	1.00	feet, NGVD 29
	Approach Apron Elevation	-5.00	feet, NGVD 29
	Weir Control	Vertical Slide	
Gates	Number of Gates	3	
	Gate Width	23.0	feet
	Gate Height	12.5	feet
	Clearance Elevation	13.50	feet, NGVD 29
	Breastwall Elevation	TBD	feet, NGVD 29
	Intermediate Pier Width	3.25	feet
Stilling Basin	Design Discharge	2,500	cfs
	Apron Elevation	-5.00	feet, NGVD 29
	Apron Length/Width	51.0/75.5	feet
	End Sill Elevation	-4.50	feet, NGVD 29
	Top of Baffle Block Elevation	-2.00	feet, NGVD 29
	Dist from crest toe to 1st row of blocks/2nd row	25.00	feet
	Velocity over End Sill	2.01	fps
	Training Wall Elevation	TBD	feet, NGVD 29
Canal Data (US/DS)	Invert - Thalweg	-7.0/-7.0	feet, NGVD 29
	Top of Bank	Varies, avg = 18.5	feet, NGVD 29
	Bottom Width	75.5	feet
	Top Width	205.0	feet
	Side Slope (V:H)	1 on 2.5	
Revetment	Riprap Extent (Downstream)	TBD	feet
	Riprap Size (D50)	TBD	feet
	Riprap Specific Weight	TBD	lb/ft3
	Max Velocity Riprap Can Withstand	TBD	fps

TABLE A-26. S-622 GATED SPILLWAY
HYDRAULIC DESIGN DATA SHEET

Location L-5 Canal south of the Griffin rock pits; x = 775,938 y = 726,388

Purpose S-622 replaces the existing plug to allow conveyance from L-6 Canal to S-8 pump station

Design Conditions	Discharge	500	cfs
	Headwater Elevation	11.93	feet, NGVD 29
	Tailwater Elevation	11.83	feet, NGVD 29
Crest Data	Shape	Ogee	
	Design Head (Hd)	6.9	feet
	Net Crest Length	45.0	feet
	Crest Elevation	5.00	feet, NGVD 29
	Approach Apron Elevation	0.00	feet, NGVD 29
	Weir Control	Vertical Slide	
Gates	Number of Gates	3	
	Gate Width	15.0	feet
	Gate Height	10.0	feet
	Clearance Elevation	11.00	feet, NGVD 29
	Breastwall Elevation	TBD	feet, NGVD 29
	Intermediate Pier Width	3.25	feet
Stilling Basin	Design Discharge	500	cfs
	Apron Elevation	0.00	feet, NGVD 29
	Apron Length/Width	33.0/51.5	feet
	End Sill Elevation	0.50	feet, NGVD 29
	Top of Baffle Block Elevation	2.00	feet, NGVD 29
	Dist from crest toe to 1st row of blocks	20.00	feet
	Velocity over End Sill	0.85	fps
	Training Wall Elevation	TBD	feet, NGVD 29
Canal Data (US/DS)	Invert - Thalweg	-5.10	feet, NGVD 29
	Top of Bank		feet, NGVD 29
	Bottom Width	50.0	feet
	Top Width		feet
	Side Slope (V:H)	1 on 2	
Revetment	Riprap Extent (Downstream)	TBD	feet
	Riprap Size (D50)	TBD	feet
	Riprap Specific Weight	TBD	lb/ft3
	Max Velocity Riprap Can Withstand	TBD	fps

TABLE A-27. S-630 PUMP STATION
HYDRAULIC DESIGN DATA SHEET

Location	L-4 Canal, east of the G-409 Pump Station		
Purpose/Operational Intent:	Water Supply Provides water to the G-409 Pump Station in order to continue deliveries to the Seminole Tribe of Florida's Big Cypress Reservation, and STAs 5 and 6 once L-4 Levee degrade is implemented.		
Design Condition:		360	cfs
Pump Station Capacity Criteria:			
Number of Pumps		4	
Pump Mix Type and Size	Electric	4@ 90	cfs
Mix Criteria:			
	1. The pump station will have 4 bays; four 90 cfs electric motor driven pumps 2. The pump mix allows for duplicate pumps throughout the system for operation and maintenance consideration		
Control		TBD	
Design Heads			
	Normal (HW=13.0 NGVD, TW=7.0 NGVD)	TBD	ft
	Maximum (HW=15.15 NGVD, TW=7.0 NGVD)	TBD	ft
Intake Water Surface Elevations			
	Maximum Non-Pumping Pumping	TBD	ft, NGVD
	Maximum Pumping	TBD	ft, NGVD
	Start Pumping	TBD	ft, NGVD
	Normal Drawdown	TBD	ft, NGVD
	Minimum Drawdown Pumping	TBD	ft, NGVD
	Minimum Non-Pumping	TBD	ft, NGVD
	Channel Invert	TBD	ft, NGVD
Discharge Water Surface Elevations			
	Maximum Non-Pumping	TBD	ft, NGVD
	Maximum Pumping	TBD	ft, NGVD
	Normal Pumping	TBD	ft, NGVD
	Minimum Pumping	TBD	ft, NGVD
	Minimum Non-Pumping	TBD	ft, NGVD
	Channel Invert	TBD	ft, NGVD

A.6.4 STRUCTURAL DESIGN

A.6.4.1 General Status of Completed and Non-Executed Efforts

Structural design of S-620, S-621, S-622, S-630, and S-8A will be completed during the design phase. During design phase the structural calculation will be completed after survey, hydraulic design, and geotechnical investigations are performed. The structural design will conform with the appropriate Engineering Manuals (EM), Engineering Regulations (ER), or Design Criteria Memorandums (DCM).

A.6.4.2 Pumping Stations

S-630 is a pump station that will be similar in design to S-357, but with the new layout of the Miller (S-488) pump station.

A.6.4.3 Overflow Spillways

S-621 and S-622 are gated structures similar to S-65EX1, using a two-phased approach offsetting the structure and existing plugs, will not require a bypass canal to be designed for construction of the structures.

A.6.4.4 Culverts

S-620 is a gated box culvert that will be designed similar to the (S-276 (C-4A)) culverts on Herbert Hover Dike (HHD).

A.6.5 MECHANICAL AND ELECTRICAL DESIGN

A.6.5.1 General

The pumping station mechanical design shall be in accordance with Hydraulic Institute Standards, EM 1110-2-3102 (General Principles of Pumping Station Design and Layout), and EM 1110-2-3105 (Mechanical and Electrical Design of Pumping Stations). The design will also follow the guidance of ETL 1110-2-313 (Hydraulic Design Guidance for Rectangular Sumps of Small Pumping Stations with Vertical Pumps and Ponded Approaches).

The seepage pumping station will have a required pumping capacity of 360 cfs.

The pump mix will be further developed during the design phase of the project, but it will likely have a mix similar to having four 90-cfs electric motor-driven pumps.

The pump intakes will likely be suction bell type. The use of formed suction intakes at the pumps shall be evaluated during preparation of the plans and specifications for the pumping station and shall be based upon the channel intake design.

Axial flow pumps will be used for the pumping station. The decision on whether the pumps will have either a conventional or siphon discharge will be determined during the preparation of the plans and specifications.

The pumping station electrical design shall be in accordance with NEC, NFPA, IESNA, TIA/IEA, IEEE, and recommended practice. Also, EM 1110-2-3102 (General Principles of Pumping Station Design and Layout) and EM 1110-2-3105 (Mechanical and Electrical Design of Pumping Stations) will be used.

Although the capacity of this station is low enough that SFWMD's Major Pumping Station Engineering Guidelines is not applicable, we will follow the applicable portions of these guidelines.

A.6.5.2 General Status of Completed and Non-Executed Efforts

Mechanical and electrical design of S-620, S-621, S-622, S-630, and S-8A will be completed during the design phase. During design phase the mechanical and electrical calculations will be completed after survey, hydraulic design, and geotechnical investigations are performed. Conceptual design used for cost assumptions.

A.6.5.3 Pumping Station S-630

The four 90-cfs pumps will be axial-flow-type vertical-shaft pumps. The pumps will be driven by direct-drive electric motors.

The pumps are expected to run at 600 rpm with an efficiency of about 80%.

The pumping station will include various support items, including the following:

a. An LPG fuel system sized to operate an emergency generator continuously for seven days.
b. Hoisting system for maintenance or repair of the pumping equipment.
c. Toilet facility with a water closet and a lavatory.
d. Kitchen-type sink.
e. Potable water system and a septic system for the plumbing fixtures.
f. Ventilation system to provide fresh air in the pump bays, generator area, and toilet room.
g. Air-conditioning system for the office.
h. Stilling wells containing float switches to be used for pump operations and water level monitoring.

A.6.5.3.1 Pumping Station Features

Station Crane/Hoist
An overhead bridge-type electric crane will be provided. The crane/hoist shall be capable of handling up to 15-ton loads. The crane/hoist will handle pumping station equipment such as the electric motor pump drive or the pump components during initial installation, as well as for general service thereafter.

LPG Generator Set
An LPG-driven generator set with capacities up to 500 kW may be provided. This generator must provide general standby power, but it also may be required to provide sufficient power to operate one of the electric motor driven pumps for as long as seven days.

Potable Water and Plumbing
A potable water supply and plumbing system will be provided. This will include a septic system. A filtered water system will be necessary for the station to supply water to a Toilet (lavatory, shower, and water closet) and small kitchen area.

Air Conditioning
Small split-system air conditioning systems will be provided for the control room, telecommunications room, and the break room.

Ventilation System
A system of air inlet openings and exhaust fans will be provided for ventilation of the operating floor area. The air inlet louvers will be the type commonly referred to as Miami-Dade louvers. Bird screening will also be provided over the openings. The wall type exhaust fans will have motor-operated dampers.

Trash Rake
Trash rake/rack system will be one of two types: an automatic, continuously rolling, flex rake and trash rack system such as that manufactured by Duperon, or a powered rail-mounted traveling trash rake and hoist car assembly with a telescoping arm used to grip and remove debris. This system is similar to ones that are manufactured by Hydro Component Systems. The system selected shall be similar to those that have proven satisfactory at previously completed pumping stations.

Pump Model Tests
The specifications will require that a series of model tests be performed to verify performance and cavitation limits of the proposed pump. The contractor will be required to construct one complete pumping system for each size pump to the necessary scale model. The pumping system will include the forebay, pump, and discharge tube. All tests for determination of compliance with guarantees of capacity and/or efficiency will be accomplished using prototype heads.

A.6.5.3.2 Electrical Features

Electric Service and Backup Generator
A 480-volt, three phase, electrical service shall be provided. The local utility company shall provide the power. Transient Voltage Surge Suppression (TVSS) shall be provided at the service entrance. A backup LPG-generator unit shall be provided to supply 480-volt, three phase electrical power when utility power is not available or not reliable. The backup generator and automatic transfer switch will be sized sufficiently to power exhaust fans, lights and SCADA equipment.

Interior Electrical Distribution
Switchgear rated for 480 volt, three phase with a main breaker will be connected to the incoming service and will feed engine control centers, motor control centers, lighting panels, power panels and station equipment defined in the Pumping Station Features above. Each engine control center will house starters and controls for auxiliary equipment for the engine unit. The main switchboard will also feed transformers to supply 120/208 or 480/277 volt loads as necessary.

Interior and Exterior Lighting
High intensity discharge, industrial high bay luminaries will be used for the main pumping station area with industrial fluorescent fixtures with electronic ballasts for office and general type areas. Exterior lighting for security purposes would be automatically controlled by photo-electric cells and contactors.

Wiring and Conduit
Insulated copper conductors will generally be installed in either PVC coated rigid galvanized steel conduit or schedule 80 rigid plastic conduit. Conductors will be rated for 600 volt insulated types XHHW or XHHW-2. All wiring will conform to UFGS Guide Specifications.

Instrumentation and Controls
The pumping station will have a centralized monitoring and control room Programmable logic controllers (PLCs) will be used to monitor and control the station auxiliaries. An Ethernet network will connect the PLCs and station computer. Ethernet based IP cameras will also connect to the Ethernet network. The station computer will allow for operation of the station via SFWMD's preferred SCADA software.

SCADA and Telemetry
The controls systems shall include manual, automatic and telemetry capabilities for the pumps and auxiliary systems. The automation components of all pumping stations and structures that will eventually be operated and maintained by South Florida Water Management District (SFWMD) must conform to SFWMD standards in order to (1) achieve cost efficiency in design, construction, and operation and maintenance, (2) meet safety, reliability, and performance requirements during routine and emergency operations. The automation components are broadly defined to include hardware, software, communications, and user interface elements.

A.6.5.4 Gated Spillways and Culverts

Gate Operators
Gate operators will be designed based on the size, weight, and hydraulic loading on the gates. The operators will either be electric motor driven through a drum and pulley system or via as an actuator on a stem screw.

Electrical Service
A control center will house a main breaker, combination starter for the gate motor, lighting panel, relay compartment, and a circuit for exterior lighting. Surge suppression will be provided for each electrical/electronic system within or outside the structure.

Control and Monitoring
Duplicate open-close push button station in the control house and at the spillway or culvert structure will be provided for manual gate control. Necessary open, close, automatic control relays, and limit switches will be incorporated in the gate control circuit. Power and control circuits for water level recorders and gate position recorders will be provided.

A.6.5.5 Weir

Water Level Indication
Stilling wells with water level monitoring equipment will be provided on both sides of the weir. Power for the monitoring equipment will be provided by either commercial power or by solar power, depending on the final location of the weir.

A.6.5.6 Telemetry

Each spillway or culvert site that requires remote automation will be equipped with an RTU compatible with the existing SFWMD telemetry system. RTU software will be in accordance with the SFWMD standard load set. The construction plans will contain plans for a fully functioning telemetry system capable of connecting to and communicating with the SFWMD existing system. Additional coordination during the development of plans and specifications will finalize the telemetry requirements.

A.7 BLUE/GREEN/YELLOW LINES – DISTRIBUTION, CONVEYANCE & SEEPAGE MANAGEMENT

A.7.1 CIVIL - SITE DESIGN

Features identified in the Recommended Plan have been designed to the level of detail necessary to provide cost estimates. Best professional judgment as well as previous project design knowledge for DECOMP DPM, MWD and L-31NSMPP was used during plan formulation alternative development and design efforts. Components of the blue, green and yellow line have been identified, sized appropriately according to available data, historic information, and best engineering judgment. All project components will be optimized during PED phase for cost efficiency and performance, incorporating updated data and information as it becomes available.

The levee in the Recommended Plan has a crown of 14 feet with one on three side slopes. The levee will be seeded to prevent erosion. All levees will have a 15 foot clear zone at the toe as required by EM 1110-2-1913, Design and Construction of Levees. For the new levee (L-67D), a waiver or variance will be requested to permit marsh type vegetation within the 15 foot clear zone. Upon completion of construction, the levee will be entered into the National Levee Data Base for regular inspections as required by P. L. 84-99 to be part of the Federal Emergency Management System. See Annex C-2 for general plate of L-67D.

A.7.1.1 General Status of Completed and Non-Executed Efforts

The following civil site project efforts remain either incomplete or were not initiated:
evaluation of alignments,
site grading,
aesthetics,
relocation of facilities,
required improvements on lands to enable proper construction of components and disposal of material,
requirements of lands for construction, operation and maintenance of the project,
identification of facility/utility relocations and methods for accomplishing relocations to include appropriate lands,
site selection and project development, and
detailed design with respect to recent Levee Safety criteria.

These analyses will be completed in PED.

A.7.1.2 Surveying Mapping Geospatial data

Historical hydrographic and topographic surveys exist for the project area. All survey data collected was performed using conventional means and methods. The existing surveys are 90-177, 90-190, 91-180, 95-116, 97-006, 98-218, 02-019, 02-037, 02-046, 02-047, 02-77, 02-142, 03-132, and 08-195. Datum's utilized for data collection is as follows: Horizontal coordinates are referenced to the State Plane Coordinate System North Atlantic Datum (NAD) 83 (2007), Florida East Zone (0901). Elevations are in US Survey Feet and referenced to North Atlantic Vertical Datum (NAVD) 88 vertical datum. See Annex C-1 for data points.

A.7.1.3 Access

Access to the project site will be from US 41 (Tamiami Trail) by S-333 along L-67A, L-67C, L-29 and L-31 levees. Northern access into WCA3B is from S-9 Pump Station or by Holliday Camp along the L67-A canal.

A.7.1.4 Material Balance and Disposal

L-67 extension (ext.) removal may place the material in the adjacent canals. Old Tamiami Trail road removal will stockpile the material within the project area, or, if suitable, the material may be used in the construction of L-67D. L-67C material may be used in the construction of L-67D or may be stockpiled adjacent to the L-67C canal. L-67D will be completed with material from onsite levee degrades (L-67C, L-67 ext removal and old Tamiami Trail road, if suitable) within the vicinity of the project area first, then from L-31 N Spoil Mound, L-29 removal or from an approved Borrow source. All peat material will be placed in either L-67 ext. or used to dress side slopes. Unsuitable material will be hauled to a certified land fill.

A.7.1.5 Utility Relocations

Florida Power and Light, and Quest Communications lines will have to be relocated where the L-29 is being removed. The removal of Old Tamiami Trial will require relocation of the Florida Power and Light line. Utilities will have to be supplied to S-631, S-632, S-633, S-333N, S-355W, and S-356.

A.7.2 GEOTECHNICAL DESIGN

Between Red and Green Lines:

<u>S-630 divide structure at western terminus of L-4</u>
a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date nor are any groundwater studies planned during the future design phase.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. It is anticipated that, the divide structure foundation will be founded on underlying limestone or compacted cohesionless fill. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. A heave or uplift evaluation will be required to design the tremie concrete slabs during the design phase. Slope stability analyses other than temporary cut slope evaluation during the design phase will not be required.

g. Excavatability analysis. Rock rippability has been estimated at this time and these estimates will be evaluated further based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the culvert and siphon structure areas can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank and through the existing embankments. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Backfill will be accomplished with compacted layers of granular backfill with rewatering. Excess excavated inorganic cobbles and grains less than 3 inches in effective diameter can be loaded onto dump trucks and hauled off the L-67D site for use as levee fill. Larger cobbles and boulders can be crushed and mixed with the minus 3 inches of soil and rock.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as backfill for the structures. Excess inorganic material will be destined to be delivered to the L-67D area for use as fill. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. Sheetpile cofferdams with a tremie concrete slab to facilitate dewatering and dry construction are typically incorporated into the construction of these type features. Discharge of dewatering effluent will be to the canal after appropriate treatment. Dewatering is typically accomplished by sump pumps within the excavation pit with supplemental groundwater lowering via well point rows. Other methods for dewatering may be utilized for construction efficiency and cost savings.

<u>Backfill Miami Canal</u>

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date nor are future groundwater studies anticipated during design.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. There is no foundation to design for with this feature. The end slopes shall be filled shallower or equal angle than 1V:3H.

g. Excavatability analysis. An excavatability analysis is not required for this feature, as it will be a fill operation.

h. Anticipated construction techniques. Some stripping of organics and vegetation off the canal side slopes will be required prior to fill placement. Organic material shall be disposed of in an area designated during the design phase. Stockpiles of borrow containing cobbles and boulders greater than 6 inches nominal diameter from the levee may need to be processed before placement in the canal. To avoid settlement, material shall be separated into pervious materials (<5% fines) for submerged fill placement and satisfactory fill for above the groundwater table in the canal. Material above the groundwater table shall be compacted and graded. Pervious materials will be placed in loose lifts starting along the canal bottom followed by compacted lifts of satisfactory fill above the groundwater table. Fill will not be allowed to be stacked on the canal slopes as this may induce slope instability.

i. Potential borrow and disposal sites. Excavated inorganic material from other feature areas will be delivered to stockpiles prior to processing in the Miami Canal. The organics are to be disposed to a designated area to be determined during the design phase.

j. Seepage and groundwater control. Seepage control other than controlled placement in wet conditions is not anticipated.

Between Green and Yellow Lines:

<u>Increase S-333 capacity by deepening or widening (S-333N)</u>

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. It is anticipated that, the pump structure foundations will be founded on underlying limestone. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. A heave or uplift evaluation will be required to design the tremie concrete slabs during the design phase. Slope stability analyses other than temporary cut slope evaluation during the design phase will not be required.

g. Excavatability analysis. Rock rippability has been estimated at this time and these estimates will be evaluated further based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the pump station structure area can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank and through the existing embankments. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Backfill will be accomplished with compacted layers of granular backfill with rewatering. Excess excavated inorganic cobbles and grains less than 3 inches in effective diameter can be loaded onto dump trucks and hauled to the L-67D levee for fill. Larger cobbles and boulders can be crushed and mixed with the minus 3 inches of soil and rock.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as backfill for the structures. Excess inorganic material will be destined to be delivered to the new L-67D levee area for fill. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. A sheetpile cofferdam with a tremie concrete slab to facilitate dewatering and dry construction is typically incorporated into the construction of this type of feature. Discharge of dewatering effluent will be to the canal after appropriate treatment. Dewatering is typically accomplished by sump pumps within the excavation pit with supplemental groundwater lowering via well point rows. Other methods for dewatering may be utilized for construction efficiency and cost savings.

<u>Two new gated structures on L-67A west of L-67D (S-632 and S-633)</u>

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date nor are future groundwater investigations anticipated.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. It is anticipated that, these spillway structure foundations will be founded on underlying limestone or compacted cohesionless fill. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. A heave or uplift evaluation will be required to design the tremie concrete slabs during the design phase. Slope stability analyses other than temporary cut slope evaluation during the design phase will not be required.

g. Excavatability analysis. Rock rippability has been estimated at this time and these estimates will be evaluated further based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the spillway structure areas can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank and through the existing embankments. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Backfill will be accomplished with compacted layers of granular backfill with rewatering. Excess excavated inorganic cobbles and grains less than 3 inches in effective diameter can be loaded onto dump trucks and hauled to the L-67D levee for use as fill. Larger cobbles and boulders can be crushed and mixed with the minus 3 inches of soil and rock.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as backfill for the structures. Excess inorganic material will be destined to be delivered to the L-67D levee for use as fill. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. Sheetpile cofferdams with tremie concrete slabs to facilitate dewatering and dry construction are typically incorporated into the construction of these features. Discharge of dewatering effluent will be to the canal after appropriate treatment. Dewatering is typically accomplished by sump pumps within the excavation pit with supplemental groundwater lowering via well point rows. Other methods for dewatering may be utilized for construction efficiency and cost savings.

One new gated structure on L-67A east of L-67D (S-631)

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. It is anticipated that, this spillway structure foundation will be founded on underlying limestone or compacted cohesionless fill. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. A heave or uplift evaluation will be required to design the tremie concrete slabs during the design phase. Slope stability analyses other than temporary cut slope evaluation during the design phase will not be required.

g. Excavatability analysis. Rock rippability has been estimated at this time and these estimates will be evaluated further based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the spillway structure area can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank and through the existing embankments. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Backfill will be accomplished with compacted layers of granular backfill with rewatering. Excess excavated inorganic cobbles and grains less than 3 inches in effective diameter can be loaded onto dump trucks and hauled to the L-67D levee for use as fill. Larger cobbles and boulders can be crushed and mixed with the minus 3 inches of soil and rock.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as backfill for the structures. Excess inorganic material will be destined to be delivered to the L-67D levee for use as fill. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. Sheetpile cofferdams with tremie concrete slabs to facilitate dewatering and dry construction are typically incorporated into the construction of these features. Discharge of dewatering effluent will be to the canal after appropriate treatment. Dewatering is typically accomplished by sump pumps within the excavation pit with supplemental groundwater lowering via well point rows. Other methods for dewatering may be utilized for construction efficiency and cost savings.

Degrade 8 Miles of L-67C between L-67D and S-333 and Export the Material

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date nor are future groundwater studies anticipated during the design phase.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. There is no foundation to design for with this feature. The end slopes shall be cut to a shallower or equal angle than currently that of the original design levee side slopes of 1V:3H. A riprap blanket with bedding may be needed on each cut face depending on design flow velocities through the gap which will be determined during the design phase.

g. Excavatability analysis. Excavatation can be conducted by standard excavating equipment of dozers, loaders, and dump trucks, as this is a degrade of an existing levee. Some excavation may be under wet conditions. No rippability evaluation is anticipated.

h. Anticipated construction techniques. Levee material will be removed by excavators and hauled off to the new L-67D levee for use as fill. Cobbles and boulders greater than 6 inches nominal diameter from the levee may need to be processed before placement in the L-67D. Riprap and bedding may be required to be placed on the end cuts of the breach for erosion protection.

i. Potential borrow and disposal sites. Excavated inorganic material will be destined to be delivered to the L-67D levee area for use as fill. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course. Riprap and bedding will be imported from offsite sources.

j. Seepage and groundwater control. There may be excavation in a wet condition and placement of riprap in the wet or dewatered condition. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration.

Degrade 6000 feet of L-67C just east of L-67D and export material

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date nor are future groundwater studies anticipated during the design phase.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. There is no foundation to design for with this feature. The end slopes shall be cut to a shallower or equal angle than currently that of the original design levee side slopes of 1V:3H. A riprap blanket with bedding may be needed on each cut face depending on design flow velocities through the gap which will be determined during the design phase.

g. Excavatability analysis. Excavatation can be conducted by standard excavating equipment of dozers, loaders, and dump trucks, as this is a degrade of an existing levee. Some excavation may be under wet conditions. No rippability evaluation is anticipated.

h. Anticipated construction techniques. Levee material will be removed by excavators and hauled off to the new L-67D levee for use as fill. Cobbles and boulders greater than 6 inches nominal diameter from the levee may need to be processed before placement in the Miami Canal. Riprap and bedding may be required to be placed on the end cuts of the breach for erosion protection.

i. Potential borrow and disposal sites. Excavated inorganic material will be destined to be delivered to the L-67D levee area for use as fill. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course. Riprap and bedding will be imported from offsite sources.

j. Seepage and groundwater control. There may be excavation in a wet condition and placement of riprap in the wet or dewatered condition. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration.

<u>Construct new levee L-67D</u>

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-28**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date. Field and laboratory hydraulic testing of the subsurface materials such as specific capacity and/or constant head recharge are to be used to estimate the horizontal hydraulic conductivity of the limestone foundation material along with double ring infiltrometer tests to estimate the vertical hydraulic conductivity.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. The levee will be founded on underlying limestone after stripping and removal of surficial organic peats and organic silts. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. Slope stability analyses during the design phase will be required for the steady state and end-of-construction cases and potentially for the rapid drawdown case. A preliminary slope stability analysis using the SLOPE/W program has been performed for the levee with results for the steady state seepage case presented in **Figure A-6**. A table of input parameters used in the analysis is contained in **Table A-28**. The computed factor of safety is 3.2.

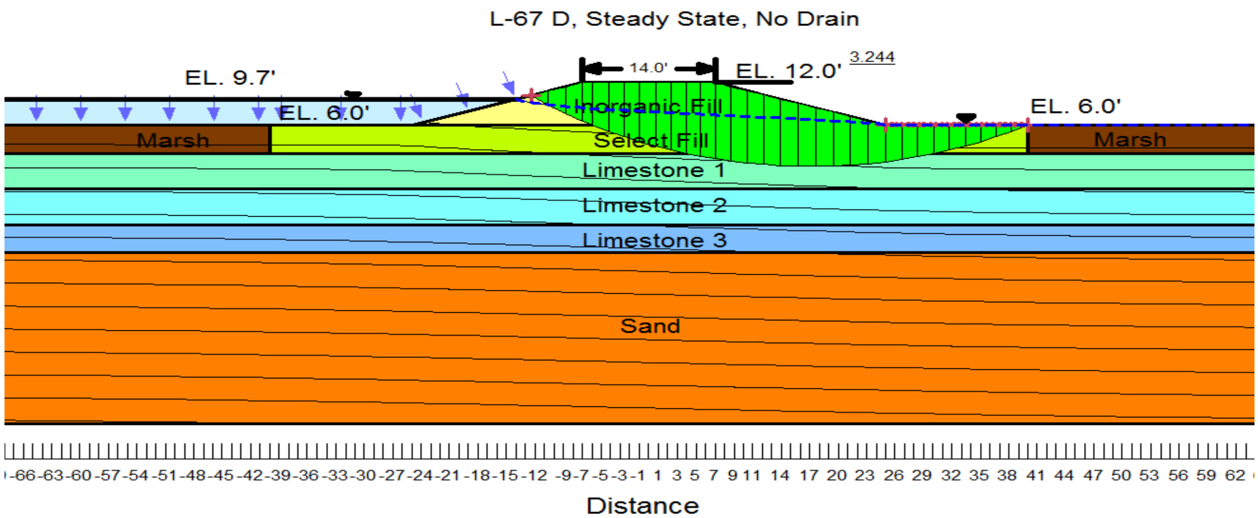


FIGURE A-6. CRITICAL CASE FOR STEADY STATE SLOPE STABILITY ANALYSIS FOR L-67D

TABLE A-28. INPUT PARAMETERS FOR THE SLOPE/W MODEL FOR L-67D

Material	Unit Weight (pcf)	Friction Angle, (°)	Elevation of Layer (ft)
Inorganic Fill	115	32	12.0 to 6.0
Marsh Organics	70	27	6.0 to 2.0
Select Fill	115	32	6.0 to 2.0
Limestone 1	120	33	2.0 to -3.0
Limestone 2	130	36	-3.0 to -7.0
Limestone 3	125	33	-7.0 to -12.0
Sand	120	32	-12.0 to -36

g. Excavatability analysis. It is anticipated that foundation materials are readily excavated with standard excavation equipment. Excavation will stop at the limestone horizon.

h. Anticipated construction techniques. It is anticipated that the levee foundation can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank. Embankment fill will be imported from many outside sources, so blending and processing will probably be required to yield a somewhat homogeneous levee. Excavated inorganic cobbles and grains less than 3 inches in effective diameter can be processed and stockpiled for use as levee fill. Larger cobbles and boulders can be crushed and mixed with the minus 6 inches of soil and rock. Excavation of the top organic layers in the foundation is anticipated to be in a wet condition. As the fill materials are primarily cohesionless, vibratory steel wheeled are pneumatic tired rollers are the expected equipment to be used on this site.

i. Potential borrow and disposal sites. Stockpiled inorganic materials from other features of this project will be processed, compacted and placed as fill for the inner and exterior dikes. Pervious fill (<5% fine grained material) will be used as the bridging lift between the underlying foundation limestone and the groundwater surface. The bridging lift will be placed loosely into the excavated pit and pushed forward with dozers until satisfactory bearing is achieved for dry placement. The pervious material can either be imported or derived from processing canal borrow material. Above the groundwater level, satisfactory fill from the canal borrow material shall be placed in compacted lifts in the dry. Oversize particles greater than 3 inches in nominal diameter shall be crushed into satisfactory fill or used for erosion protection features. Excess inorganic material will be stored in a designated disposal area which will be identified during the design phase. The organic soils are to be disposed to a designated area to be determined during the design phase. These may be used later for mixing and seeding of embankment erosion protection. Riprap and bedding materials required for erosion protection will be obtained from offsite sources unless suitable rock is found on-site. Rip rap and bedding will be sized during the design phase based on design water velocities. Access roads and the levee crest road will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. From previous studies and core boring logs from L-67A/C to the west, solution cavities are expected to be encountered in foundation grade for the levee. Field and laboratory tests are planned to characterize the subsurface material hydraulic characteristics for the purpose of minimizing seepage losses and to protect against piping losses of embankment material from beneath the levee foundation. A preliminary 2D seepage analysis was performed on an idealized levee cross section using the finite element method contained in the SEEP/W software. The numeric model is shown in **Figure A-7**. Hydraulic material properties used in the analysis are presented in **Table A-29.** The levee geometry is based on the Recommended Plan idealized section. Maximum headwater was modeled at elevation 9.7' NAVD88 with the tailwater fixed at the ground surface elevation of 6' NAVD 88. Seepage quantities through the levee under these hydraulic loading conditions indicate underseepage at about 18 cubic feet per day. From previous investigations it is anticipated that the levee will be founded on weathered limestone which may contain vugs and solution cavities. If the cavities are large enough, some sort of filter may be required between the embankment and the foundation to prevent piping of the embankment materials. Geologic mapping of the levee foundation using conventional methods would be impractical in a wet environment. Stochastic mapping of rock core voids and/or geophysical methods may be required to estimate the pore size distribution of the limestone foundation.

Blue Line and South:

<u>Gated Spillway just east of L-67D (S-355W)</u>

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date nor are any groundwater studies planned during the future design phase.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. It is anticipated that, the spillway structure foundation will be founded on underlying limestone or compacted cohesionless fill. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. A heave or uplift evaluation will be required to design the tremie concrete slabs during the design phase. Slope stability analyses other than temporary cut slope evaluation during the design phase will not be required.

TABLE A-29. INPUT PARAMETERS FOR SEEP/W MODEL FOR L-67D

Material	Anisotropy, K_v/K_h	Saturated Hydraulic Conductivity, K_h (ft/day)	Top and Bottom Elevation of Layer (ft)
Inorganic Fill	0.25	0.3	12.0 to 6.0
Marsh Organics	0.7	33	6.0 to 2.0
Select Fill Sand	1	50	6.0 to 2.0
Limestone 1	0.5	11	2.0 to -3.0
Limestone 2	0.33	63	-3.0 to -7.0
Limestone 3	0.5	11	-7.0 to -12.0
Sand	0.5	11	-12.0 to -36

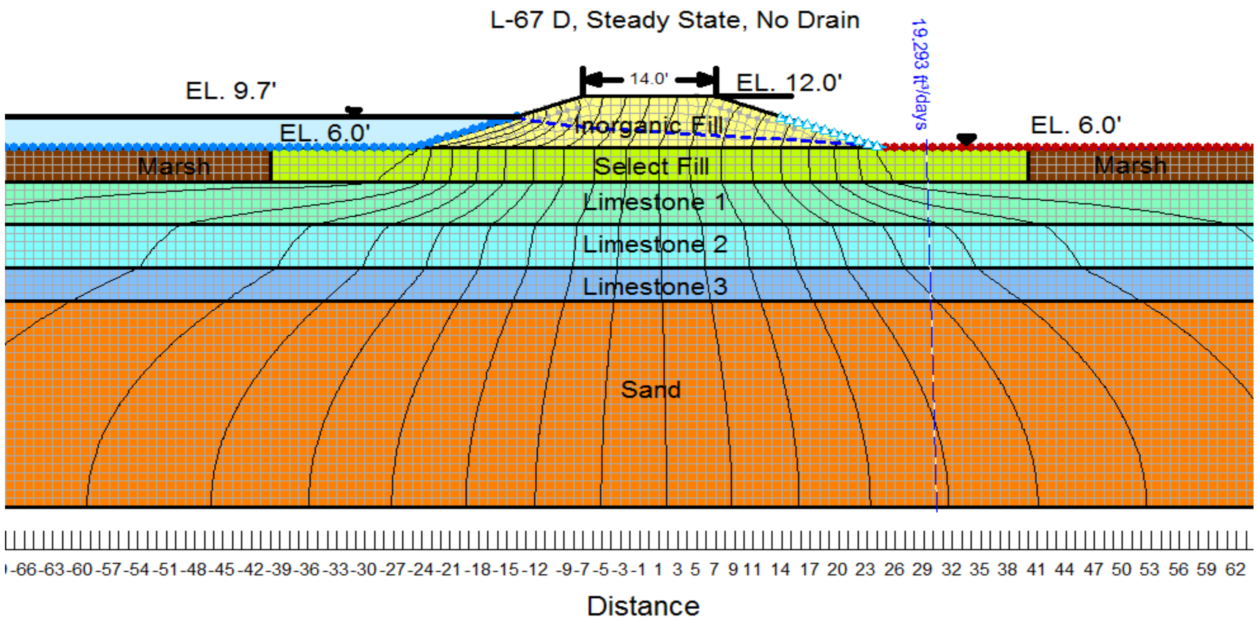


FIGURE A-7. PRELIMINARY STEADY STATE SEEPAGE ANALYSIS OF L-67D

g. Excavatability analysis. Rock rippability will be evaluated based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the culvert and siphon structure areas can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank and through the existing embankments. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Backfill will be accomplished with compacted layers of granular backfill with rewatering. Excess excavated inorganic cobbles and grains less than 6 inches in effective diameter can be loaded onto dump trucks by rubber-tired loaders and hauled off the L-67D site for use as levee fill. Larger cobbles and boulders can be crushed and mixed with the minus 6 inches of soil and rock.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as backfill for the structures. Excess inorganic material will be destined to be delivered to the L-67D area for use as fill. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. A sheetpile cofferdam with a tremie concrete slab to facilitate dewatering and dry construction is typically incorporated into the construction of these type features. Discharge will be to the canal after appropriate treatment. Dewatering is typically accomplished by sump pumps within the excavation pit with supplemental groundwater lowering via well point rows. Other methods for dewatering may be utilized for construction efficiency and cost savings.

<u>Remove L-67 Extension and backfill adjacent canal</u>

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date nor are any groundwater studies planned during the future design phase.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Per paragraph A.6.2.e above, earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. There is no foundation to design for with this feature. The end slope shall be cut to a shallower or equal angle than currently that of the original design levee side slopes of 1V:3H. A riprap blanket with bedding may be needed on each cut face depending on design flow velocities through the gap which will be determined during the design phase.

g. Excavatability analysis. Excavation can be conducted by standard excavating equipment of dozers, loaders, and dump trucks, as this is a degrade of an existing levee. Some excavation may be under wet conditions. No rippability evaluation is anticipated.

h. Anticipated construction techniques. Levee material will be removed by excavators and placed in the adjacent borrow canal. Cobbles and boulders greater than 6 inches nominal diameter from the levee may need to be processed before placement in the canal. To avoid settlement, material shall be separated into pervious materials (<5% fines) for submerged fill placement and satisfactory fill for above the groundwater table in the canal. Material above the groundwater table shall be compacted and graded. Riprap and bedding may be required to be placed on the end cuts of the breach for erosion protection.

i. Potential borrow and disposal sites. Excavated inorganic material will be delivered to the adjacent borrow canal. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Riprap and bedding will be imported from offsite sources.

j. Seepage and groundwater control. There may be excavation in a wet condition and placement of riprap in the wet or dewatered condition. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration.

Remove Six miles of Tamiami Trail Road from L-67 Extension to Tram Road

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date nor are any groundwater studies planned during the future design phase.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. There is no foundation to design for with this feature. The end slopes shall be cut to a shallower or equal angle than currently that of the original design levee side slopes of 1V:3H. A riprap blanket with bedding may be needed on each cut face depending on design flow velocities through the gap which will be determined during the design phase.

g. Excavatability analysis. Excavatation can be conducted by standard excavating equipment of dozers, loaders, and dump trucks, as this is a degrade of an existing levee. Some excavation may be under wet conditions. No rippability evaluation is anticipated.

h. Anticipated construction techniques. Levee material will be removed by excavators and hauled off to the new L-67D levee for use as fill or placed in a designated disposal area which will be established during the design phase. Cobbles and boulders greater than 6 inches nominal diameter from the levee may need to be processed before placement on the L-67D levee. Riprap and bedding may be required to be placed on the end cuts of the breach for erosion protection.

i. Potential borrow and disposal sites. Excavated inorganic material will be destined to be delivered to the L-67D levee area for use as fill or to a designated disposal site. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Riprap and bedding will be imported from offsite sources.

j. Seepage and groundwater control. There may be excavation in a wet condition and placement of riprap in the wet or dewatered condition. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration.

<u>Increase S-356 pump station capacity by deeper pumps or lateral extent.</u>

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. It is anticipated that, the pump structure foundations will be founded on underlying limestone. These foundation materials are typically adequate in regards to bearing capacity and settlement. A bearing capacity evaluation and settlement analysis will be performed during the design phase after collection of geotechnical exploration data during the design phase. A heave or uplift evaluation will be required to design the tremie concrete slabs during the design phase. Slope stability analyses other than temporary cut slope evaluation during the design phase will not be required.

g. Excavatability analysis. Rock rippability will be evaluated based on available engineering design and construction records and new test pits during the geotechnical exploration activities during the design phase for this feature.

h. Anticipated construction techniques. It is anticipated that the pump station structure area can be excavated by standard hydraulic excavator within the layers of peat and organic materials in the top 4-6 feet from the bank and through the existing embankments. Below the lowest level of these materials excavators with ripping buckets should be able to break through the underlying limestone layers. For harder rock, pneumatic picks and/or blasting may be required to remove unrippable rock strata. Backfill will be accomplished with compacted layers of granular backfill with rewatering. Excess excavated inorganic cobbles and grains less than 6 inches in effective diameter can be loaded onto dump trucks by rubber-tired loaders and hauled off to the L-67D levee for fill or to a designated disposal area. Larger cobbles and boulders can be crushed and mixed with the minus 6 inches of soil and rock.

i. Potential borrow and disposal sites. Excavated inorganic materials will be processed, compacted and placed as backfill for the structures. Excess inorganic material will be destined to be delivered to the new L-67D levee area for fill or to a designated disposal area. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Access roads will be surfaced with a minimum of 6 inches of limestone base course.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration. A sheetpile cofferdam with a tremie concrete slab to facilitate dewatering and dry construction is typically incorporated into the construction of this type of feature. Discharge will be to the canal after appropriate treatment. Dewatering is typically accomplished by sump pumps within the excavation pit with supplemental groundwater lowering via well point rows.

j. Seepage and groundwater control. A dewatering evaluation will be performed with seepage analysis during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration.

<u>Degrade 4.3 miles of L-29 in Blue Shanty flow way</u>

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date nor are any groundwater studies planned during the future design phase.

d. Recommended Instrumentation. Geotechnical instrumentation is not forecasted for this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. There is no foundation to design for with this feature. The end slopes shall be cut to a shallower or equal angle than currently that of the original design levee side slopes of 1V:3H. A riprap blanket with bedding may be needed on each cut face depending on design flow velocities through the gap which will be determined during the design phase.

g. Excavatability analysis. Excavatation can be conducted by standard excavating equipment of consisting of dozers, loaders, and dump trucks as this is simply the degrading of an existing levee. Some excavation may be under wet conditions. No rip ability evaluation is anticipated.

h. Anticipated construction techniques. Levee material will be removed by excavators and hauled off to the new L-67D levee for use as fill or placed in a designated disposal area which will be established during the design phase. Cobbles and boulders greater than 6 inches nominal diameter from the levee may need to be processed before placement on the L-67D levee. Riprap and bedding may be required to be placed on the end cuts of the breach for erosion protection.

i. Potential borrow and disposal sites. Excavated inorganic material will be destined to be delivered to the L-67D levee area for use as fill or to a designated disposal site. The organics are to be disposed to a designated area to be determined during the design phase. Riprap and bedding materials required for erosion protection will be obtained from offsite sources and will be sized during the design phase based on design water velocities. Riprap and bedding will be imported from offsite sources.

j. Seepage and groundwater control. There may be excavation in a wet condition and placement of riprap in the wet or dewatered condition. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase geotechnical exploration.

<u>Partial depth seepage cutoff wall along L-31N with no new canals</u>

a. Selection of preliminary design parameters. The preliminary geotechnical design parameters for this project are established based on typical values for similar materials on COE projects in South Florida, empirical relationships form literature and from data from previous projects in the study area. The tentative design parameters are presented in **Table A-20**.

b. Geophysical Investigations. No geophysical investigations have been performed in the vicinity of this feature to date.

c. Groundwater Studies. No groundwater studies have been performed in the vicinity of this feature to date.

d. Recommended Instrumentation. Vibrating wire piezometers upstream and downstream of the cutoff wall may be required as part of the design of this feature.

e. Earthquake Studies. Earthquake studies will not be required for this feature of work due to the extremely low seismicity of South Florida.

f. Preliminary foundation design and slope stability analyses. No bearing capacity is required for this feature. A settlement evaluation will be conducted on a soil-cement-bentonite wall, if used, during the design phase. A slope stability analysis will be performed during the design phase because the wall will change the hydraulic loading conditions on the embankment of L-31N.

g. Excavatability analysis. Rock rippability will be not be evaluated explicitly as the two candidate wall types are a vinyl sheetpile or SBC wall. Soft rock can be driven through with driving shoes and harder rock can be broken up with a chisel beam. The SBC wall installation techniques such as the Trench-cutting and Remixing Deep (TRM), the Cutter Soil Mix (CSM) and the hydromill methods are capable of grinding the rock into soil so that it can be integrated in the wall mix as it is being pulverized.

h. Anticipated construction techniques. The SBC wall installation techniques such as the Trench-cutting and Remixing Deep (TRM), the Cutter Soil Mix (CSM) and the hydromill methods can be used to install the cutoff wall. All three walls method types have been used successfully at the Herbert Hoover Dike project. They all involve to one extent or another in-situ mixing of subsurface materials with injected bentonite, cement and water to form portions of a continuously formed wall along the levee alignment. The vinyl sheetpile method can also be used. A cost analysis should be performed to select the most cost effective option. The SBC walls need a sufficient source of water during production. Prior to production installation of the wall, a test panel should be constructed and evaluated for compliance with the performance criteria. Periodic testing along the wall during production is essential to assure compliance with specified permeability, uniformity and strength criteria.

i. Potential borrow and disposal sites. Soils materials for the SBC mixes are taken from the embankment rock and soil. If there is an excess of organic material in the wall area subsurface materials, replacement select fill can be imported for replacement of the organic soils. Cement and bentonite materials are obtained from outside sources. Water is to be obtained from the adjacent canals, wells or can be imported, if necessary. Excess and waste materials will be placed in a designated disposal area which will be identified during the design phase.

j. Seepage and groundwater control. As a minimum, a 2D seepage analysis will be conducted during the design phase. Sufficient hydraulic conductivity data from specific capacity tests, lab permeability tests, constant head recharge tests, and/or slug tests will be conducted during design phase of geotechnical exploration. Wall water tightness during construction will be continually checked against the permeability criteria. Piezometers will be installed during construction to monitor the phreatic surface before and after installation of the wall. A 3D seepage numeric model of the wall and levee system is recommended during the design phase to quantify acceptable seepage losses under around the ends of the wall system and establish wall permeability criteria and establish wall dimensions.

k. Summary of additional geotechnical exploration for the L-31N partial cutoff wall. Extremely limited or minimal geotechnical data is available at this location in this project. This cutoff wall will be constructed, in part, below the groundwater table. The wall shall receive a core boring at the crest and at the embankment toe on a spacing of tentatively 1000 feet. These core borings will consist of standard split spoon sampling and auger drilling in soil and rock core barrel drilling in rock. At least one boring shall be deep enough to identify subsurface layers and to establish the hydrogeologic and physical properties of underlying strata for modeling purposes. Undisturbed Shelby tube samples shall be obtained for cohesive materials encountered during drilling. Laboratory index tests for soil and rock will be performed on samples obtained from the drilling. Waxed rock core samples shall be obtained to determine the rock strength parameters. Companion borings will be drilled alongside the core borings to conduct field hydraulic tests of the underlying strata. Laboratory tests shall also be performed on remolded samples to determine the vertical permeability of soils. In place field hydraulic tests will include specific capacity, constant head recharge and possibly slug tests.

A.7.2.1 General Status of Completed and Non-Executed Efforts

Summary of additional geotechnical exploration for all CEPP levee degrading and canal backfilling. Extremely limited or minimal geotechnical data is available for the site specific location of the degrading and canal backfilling features in this project. However, embankment degrading will require only enough exploration to confirm the character of the embankment materials for bidding purposes. For canals probes within the canal bottom should be sufficient to characterize the canal bottom muck and underlying sands or rock to minimize subsidence of the backfill. Laboratory index tests for soil and rock will be performed on samples obtained from the drilling.

Summary of additional geotechnical exploration for the L-67D levee. Extremely limited or minimal geotechnical data is available for the site specific location of the levee. Much of the available subsurface data was obtained outside the footprint of levee. It is foreseen that the surficial organic materials under the levee footprint will be removed and replaced with a pervious fill bridging lift below the groundwater table. Therefore, geotechnical explorations need to encompass dewatering features in addition to data required for facilities constructed at ground level. The levee embankment will receive a minimum core boring at the crest and toe at a tentative spacing of 1000 feet. These borings shall consist of standard split spoon sampling and auger drilling in soil and rock core barrel drilling in rock. Borings shall be deep enough to identify suitable soft layers in the foundation and establish the hydro geologic properties of underlying strata for modeling purposes. Undisturbed Shelby tube samples shall be obtained for cohesive materials encountered during drilling. Laboratory index tests for soil and rock will be performed on samples obtained from the drilling. Waxed rock core samples shall be obtained to determine the rock strength parameters. Companion borings will be drilled alongside the core borings to conduct field hydraulic tests of the underlying strata. Laboratory tests shall also be performed on remolded samples to determine the vertical permeability of soils. In place field hydraulic tests will include specific capacity, constant head recharge and possibly slug tests. Samples from stockpiles from other project features will be of a wide range of particle distributions. Excavated stockpiled material will have laboratory index testing performed during the design phase to determine fill processing requirements. Geophysical testing may be used to identify and map solution cavities in the levee footprint. A test fill for embankment construction feasibility in the wet and a breakdown analysis of excavated limestone and soil after compaction will also be evaluated during the exploration program. Rock core samples may be mapped by digital photography to stochastically estimate porosity, permeability and filtration characteristics of limestone layers.

Summary of additional geotechnical exploration for the L-31N partial cutoff wall. Extremely limited or minimal geotechnical data is available at this location in this project. This cutoff wall will be constructed, in part, below the groundwater table. The wall shall receive a core boring at the crest and at the embankment toe on a spacing of tentatively 500 feet. These core borings will consist of standard split spoon sampling and auger drilling in soil and rock core barrel drilling in rock. At least one boring shall be deep enough to identify subsurface layers and to establish the hydrogeologic and physical properties of underlying strata for modeling purposes. Undisturbed Shelby tube samples shall be obtained for cohesive materials encountered during drilling. Laboratory index tests for soil and rock will be performed on samples obtained from the drilling. Waxed rock core samples shall be obtained to determine the rock strength parameters. Companion borings will be drilled alongside the core borings to conduct field hydraulic tests of the underlying strata. Laboratory tests shall also be performed on remolded samples to determine the vertical permeability of soils. In place field hydraulic tests will include specific capacity, constant head recharge and possibly slug tests.

A.7.2.2 Soils

The area of the Blue Line, where it borders Miami-Dade County, is the Biscayne Gravelly Marl (U.S.Department of Agriculture 1996). Soils of this type are poorly drained and situated on broad, low flats, in sloughs, and in transverse glades that extend from the Pineland Ridge. Typically, the surface layer is about 7 inches of dark gray gravelly marl that has a texture of silt loam.

The agricultural areas of the Yellow Line (Miami-Dade and Broward Counties) (as well as the Blue Line), it is common to encounter “mixed” soils called “rock plowed” soil such as Chekika and Krome (U.S. Army Corps of Engineers, 2009c). This soil is a manmade material created by farmers excavating and crushing the soft underlying Miami Limestone, and mixing/tilling it along with the natural overburden soils. Consequently, the overburden thickness is somewhat higher in these areas. In most cases, the underlying Miami Limestone controls the infiltration of rain or introduced stormwater due to the high permeability of the rock-plowed soils.

A.7.2.3 Geology

Rocks formed in a shallow marine depositional environment under tropical and subtropical environmental conditions exist in the Blue Line area. Three geological formations comprise this sedimentary package: the Pamlico Sand, the Miami (Oolite) Limestone, and the Fort Thompson Formation. This sedimentary package rests unconformably on quartz sands of the Pliocene Tamiami Formation, which serves as basement for this area. The thickness of the Quaternary marine sediment package increases north to south from approximately 40 feet at the Red Line to approximately 100 feet at the Blue Line.

<u>Blue Line</u>
The oolitic Miami Limestone, the upper portion of the Biscayne Aquifer, often outcrops at the surface near the Blue Line and forms approximately 10 to 15 feet of caprock overlying the Fort Thompson Formation. The Blue Line is characteristic of karstic marine carbonates of the Fort Thompson Formation.

In 1950, a series of 3-inch diameter core borings were drilled at depths 18 to 35 feet a one or two-mile centers along the L-29 (the Blue Line Boundary). Geology shows peat and/or organic sediments are between one or two feet thick. A marl underlies these sediments. The marl ranges in thickness from one to two feet thick. Below the marl is a limestone which ranges in thickness from approximately 7 to 45 feet thick. Below the limestone layer is some interbeds of shell and sand (U.S. Army Corps of Engineers, 1951).

Recharge tests were made simultaneously with many of the core borings along the levee alignment of L-29 (U.S. Army Corps of Engineers, 1951). The results of the tests typically ranged from 2-3 gallons per minute (gpm) on the western side of the levee to 40-84 gpm on the eastern side. Borings logs with recharge test data can be found in the 1951 U.S. Army Corps of Engineers report.

Green Line

Previously, local geologic investigations of the L-67A and L-67C have been completed by the U.S. Army Corps of Engineers Jacksonville District (ANNEX G-3). The 14 test pit excavations in 2002 (U.S. Army Corps of Engineers, 2002) to the top of limestone show three distinctive lithologies in the L-67 area from top to bottom: fill material of gravel to cobble-sized limestone up to 10 feet thick, peat up to 4 feet thick, and hard limestone.

A subsequent 4-boring investigation of the L-67 area by the U.S. Army Corps of Engineers in 2009b as part of the DECOMP Phase I study indicates basically the same three lithologies mentioned above except the peat appeared to be embedded in a silt layer below the fill material.

Yellow Line

Several U.S. Army Corps of Engineers geological investigations have been conducted of the L-30 and L-31N to address seepage management issues in these areas as well as to characterize the subsurface lithology (ANNEX G-3). These include the 2006 L-30 seepage management pilot project, the 2008 follow-up L-30 seepage geotechnical data report, and the 2011 Plans and Specifications for the construction of the L-31N seepage barrier.

Challenge Engineering and Testing performed three initial borings in 2006 for the L-30 area for the U.S. Army Corps of Engineers (Challenge Engineering and Testing, 2006). Results of the boring investigation indicate thick fill material overlies a massive limestone interpreted to be the Fort Thompson Formation. Below the limestone are interbeds of shell, sand, and minor clay.

Wolf WPC 2008 performed three additional borings and came up with similar lithologies like those of Challenge except minor peat and clay layers are situated between the fill material and the limestone.

Finally, AMEC 2011 prepared the L-31N Seepage Barrier Phase I Plans and Specifications which they show some cross-section lithology where a section of L-31N runs into the Tamiami Trail roadway. The cross-sections show fill resting on top of limestone (the limestone varies from oolitic to dense to marine to freshwater).

A.7.2.4 HTRW

The Corps will review the HTRW condition of the affected parcels and ensure that the proper due diligence is performed in accordance with ER 1165-2-132 prior to certifying lands for construction. Should remediation of HTRW contamination be required, it is the responsibility of the SFWMD, the non-Federal, sponsor and is not a creditable cost to the project.

A.7.3 HYDRAULIC DESIGN

A.7.3.1 General Status of Completed and Non-Executed Efforts

A.7.3.2 Hydraulic Design - General

This section provides a brief overview of the hydraulic design criteria, parameters, intent/purpose of project features. Detailed hydraulic design of individual components is described in later sections, including hydraulic design data sheets. Currently, all elevations are referenced to NGVD 29; elevations will be provided in both NGVD 29 and NAVD 88 when revised during PED.

A.7.3.2.1 Design Criteria and Parameters

This section includes criteria and parameters that were used in the hydraulic design of the Blue/Green/Yellow line components.

A.7.3.2.1.1 Head Loss

Due to the relatively flat topography throughout the project area, the hydraulic head losses across many of the control structures are low, resulting in the design of larger structures (number and size of barrels, bays, etc.) than may typically be assumed for other regions. The use of pumps was avoided wherever possible to reduce operation and perpetual maintenance costs. During PED phase, SAJ expects to optimize system operations and therefore structure sizes for cost and performance efficiencies.

A.7.3.2.1.2 Flow and Velocity

Design flow rates for all water control structures were determined based on RSM-BN model outputs and existing canal and structure capacities. To capture cost impact adequately, structures and canals were designed for maximum capacity scenarios. Optimization of these features will be conducted during the PED phase for performance and cost efficiency.

A.7.3.2.1.3 Water Control Structures

The proposed plan for the area encompassed by the blue/green/yellow lines includes S-631, S-632, and S-633 culverts, and S-333N and S-355W spillways. The function of the control structures is to provide a direct hydrologic connection between WCA-3A, WCA-3B, and Northeast Shark River Slough. These structures will improve hydrologic conditions in WCA-3A, WCA-3B, and ENP by improving the timing and spatial distribution for the increased WCA-3A inflows provided by CEPP, while also providing increased outlet capacity to help manage WCA-3A high water events more effectively. The S-631, S-632, and S-633 are duplicate structures along the L-67A Canal, with S-631 north of the proposed L-67D Levee, and S-632 and S-633 south of the levee within the CEPP flowway. The design capacity for all three culverts is 500 cfs. The S-333N gated spillway was designed to operate in conjunction with the existing S-333 spillway for a total capacity of 2,500 cfs. The purpose of this expansion was to increase deliveries into the ENP Northeast Shark River Slough and to provide increased outlet capacity for southern WCA-3A. The S-355W spillway was designed to convey flows from the west to east within L-29 Canal with the intent of aiding in meeting ecological objectives in Everglades National Park by maintaining water deliveries to the eastern MWD 1-mile bridge. The spillway was sized to 1230 cfs to match the conveyance capacity of the existing S-334 spillway to the east and in ENP.

A.7.3.2.1.3.1 Gated Culverts

The entire CEPP project includes numerous gated box culverts across the entire project area. Construction material for all culverts is to be cast in place concrete.

An entrance loss coefficient value of 0.9 (assumed due to gate-added turbulence around inlet) and exist loss coefficient of 1.0 was used for all gated culvert structures. Also, the Manning's friction or energy loss coefficient was assigned 0.013 for all culverts. All major conveyance culverts were designed to remain submerged year round to reduce aquatic growth within, thereby better maintaining design friction head losses. All gated culverts sites were designed with a minimum of two culverts to allow maintenance activities to coincide, however with reduced capacity operations.

A.7.3.2.1.3.2 Gated Spillways

The CEPP Recommended Plan for the blue/green/yellow lines includes the design of two ogee weir concrete spillways with steel vertical lift gates. The spillways were designed to be in conformity of engineering guidance found in USACE EM-1110-2-1603. All ogee spillways have vertical gates for controlled discharge operations. The spillways were designed with minimal head differential for conveyance energy. The S-355W spillway was designed with 1.0 foot head differential. The S-333N spillway head differential was chosen to match that of the existing S-333 spillway, at 0.5 feet.

A.7.3.2.1.3.3 Pump Stations

The CEPP Recommended Plan proposes to replace the existing temporary S-356 pump station with a permanent structure. The current temporary structure has a capacity of 575 cfs. The design condition of the new structure will be increased to 1,000 cfs. The design capacity will be increased to 1,650 cfs to accommodate design requirements and design redundancy. S-356 will be a four-bay pump station with three 500 cfs diesel engine driven pumps (one redundant) and one 150 cfs electric motor driven pump, for a total capacity of 1,650 cfs. The S-356 provides seepage management for increased CEPP stages in eastern WCA-3B and eastern ENP, with return to Northeast Shark River Slough.

A.7.3.2.1.4 Levees

The proposed plan includes a new levee, L-67 D (Blue Shanty Levee) to be constructed between the L-29 and L-67A Canals to provide a boundary for the Blue Shanty Flowway. The levee is designed for a top of bank height of 12.00 ft NGVD.

Detailed design analysis for hydraulic components along the Blue/Green/Yellow lines can be found in supplemental documents located in Appendix A, Annex A-1.

A.7.3.3 Blue/Green/Yellow Lines – Distribution, Conveyance & Seepage Management

A.7.3.3.1 General Information

A.7.3.3.1.1 Purpose

The purpose of the project features along the blue/green/yellow lines is to provide additional distribution and conveyance to areas in WCA-3B and ENP. The installation of the proposed L-67D Levee, S-632, S-633 culverts, and the partial degrade of the L-29 Levee in the adjacent area will hydrologically connect WCA-3A to these southern areas by way of a new flowway (Blue Shanty Flowway). At times when capacity is available, the proposed S-333N and existing S-333 spillways will provide additional conveyance to ENP. S-631 is located north of the L-67D Levee in order to provide conveyance to eastern WCA-3B to improve hydroperiods. The S-355W spillway will convey water from the Blue Shanty Flowway, S-333, and S-333N eastward toward the existing S-334 spillway when needed to provide conveyance to meet ecological objectives.

A.7.3.3.1.2 Location

The proposed CEPP features encompassed by the blue/green/yellow lines lie within Miami-Dade County. The S-631, S-632, and S-633 gated culverts are located on the southern portion of the L-67A Canal, with S-333N south of the intersection of the L-67A and L-67C Canals. The proposed L-67D Levee is located between the L-67A and the L-29 Canals, eastward of the L-67 Extension. The S-355W spillway is located in the L-29 Canal at the intersection of the Blue Shanty Levee and the L-29 Canal.

A.7.3.3.1.3 Features

The CEPP project has the following hydraulic features within the blue/green/yellow line boundaries:

Structures:
- S-631 Gated Culvert
- S-632 Gated Culvert
- S-633 Gated Culvert
- S-333N Gated Spillway
- S-355W Gated Spillway
- S-356 Pump Station

Levees:
- L-67D Blue Shanty Levee

Figure A-8 illustrates all feature locations for the Blue/Green/Yellow Line areas (structures and canals are not to scale or geographically referenced). Design analysis for the gated spillways within the blue, green, and yellow line boundaries can be found in supplemental documents located in Appendix A, Annex A-1.

FIGURE A-8. BLUE/GREEN/YELLOW LINE FEATURE LOCATION MAP

A.7.3.3.2 Hydraulic Design

A.7.3.3.2.1 Proposed Water Control Structures

A.7.3.3.2.1.1 Gated Culverts

S-631, S-632, and S-633 Gated Culverts

The S-631, S-632 and S-633 structures are identical gated culverts used for increased conveyance from WCA-3A to WCA-3B through the L-67A Levee and proposed gapping in L-67C Levee. The three structures are located along the southern portion of the existing L-67A Canal, with S-631 located north of the proposed L-67D Levee and S-632 and S-633 located south of L-67D. The three structures are single-barreled gated box culverts with dimensions of 11 ft by 11 ft each with vertical slide gates, and a total length of 100 feet. The upstream and downstream inverts were determined based on historical low TW elevations (7.50 ft NGVD in the gap between L-67A and L-67C Canals) and adjusted to accommodate the height of the barrel plus one foot of clearance. The upstream and downstream inverts were set to -4.50 ft NGVD. Since the barrel inverts are below the bottom of canal elevation, the canal will taper at a slope of 1V:5H to match grade. The design flow for the three structures is 500 cfs with a design hydraulic head of 0.5 feet. The design velocity through the structures is 4.13 fps.

A.7.3.3.2.1.2 Gated Spillways

S-333N Gated Spillway

The S-333N structure is a gated spillway structure that will work in conjunction with the existing S-333 spillway to increase hydraulic connectivity between WCA-3A and Everglades National Park. S-333N is a complimentary structure to the existing S-333 spillway. The structure is a one-bay gated spillway with a design flow of 1,150 cfs and hydraulic head of 0.5 feet. The design flow was established in order to reach a combined conveyance with the existing S-333 spillway (1,350 cfs) of 2,500 cfs. The spillway gate is 29 ft wide by 14.6 ft high, with a crest elevation at -3.10 ft NGVD. The upstream and downstream apron elevations are set at -6.00 ft NGVD with apron lengths and widths of 39.0 ft and 29.0 ft, respectively. The S-333N structure is located south of the intersection of L-67A and L-67C Canals.

S-355W Gated Spillway

The S-355W structure is a gated spillway in the L-29 Canal, located at the southern extent of the proposed L-67D levee. The purpose of the S-355W is to convey water from the L-29 Canal within the Blue Shanty Flowway, eastward towards the existing S-334 spillway to provide assistance in meeting ENP ecological objectives. The structure is a three-bay gated spillway with a design capacity of 1,230 cfs and hydraulic head of 1.0 foot. The design flow was set to match the capacity of S-334. The spillway consists of three bays with dimensions of 12 ft wide by 8 ft high. The crest invert elevation is set to 4.00 ft NGVD. The upstream and downstream aprons are set at elevation -4.00 ft NGVD with a width and length of 36.0 feet and 42.5 feet, respectively.

A.7.3.3.2.1.3 Pump Stations

The CEPP Recommended Plan proposes to replace the existing temporary S-356 Pump Station with a permanent structure. The current temporary structure has a capacity of 575 cfs. The capacity of the new structure will be increased to 1,000 cfs, with a total pumping capacity of 1,650 cfs including design redundancy. The S-356 provides seepage management for increased CEPP stages in eastern WCA-3B and eastern ENP, with return to Northeast Shark River Slough.

Pump Rates

The pump station is designed to work in conjunction with the proposed seepage barrier south of the Tamiami Trail along L-31N. The pumping rate of 1,000 cfs was determined based on CEPP screening and formulation efforts to prevent increased flooding impacts. The collected seepage will be returned to Northeast Shark River Slough.

Pump Mix

Pump mixes were based on a minimum of two bay pump stations to minimize risk of impact to private lands should a single pump fail during critical times. All small pump stations will be equipped with electric motor driven pumps that have diesel generators or pumps for an alternative power source in cases of power outages. One criterion for all pump mixes was to utilize duplicate pump sizes as much as possible to reduce operation and maintenance costs. This is accounted for through a reduction in different spare parts required and focusing mechanical expertise. Another criterion was to provide a pump mix that allows a smooth pump rate change interval from start-up to full capacity. The S-356 is designed to be a four-bay pump station with three 500 cfs diesel engines and one 150 cfs electric motor driven pump. The design condition of 1,000 cfs will be achieved with two 500 cfs diesel engine driven pumps, with one 500 cfs diesel engine to serve as a redundant pump unit, per SFWMD Major Pumping Station Engineering Guidelines.

<u>Pump Stages</u>

Pump stages were defined by the following pumping parameters:

Intake Water Surface Elevations

Maximum Non-Pumping: Highest canal or pool stage that can be expected to occur.
Maximum Pumping: Maximum canal or pool stage that can be pumped with any increase in stage requiring the pump to be turned off. In most cases, Maximum Non-Pumping and Maximum Pumping stages are identical.
Start Pumping: Canal or pool stage when pump may be turned on as defined by system conditions, typically on the increasing limb.
Normal Drawdown: Expected local drawdown at the pump station intake.
Minimum Drawdown: Lowest local drawdown stage before pump is required to be turned off.
Minimum Non-Pumping: Lowest canal or pool stage that can be expected to occur under non-pumping conditions.

Discharge Water Surface Elevations

Maximum Non-Pumping: Highest canal or pool stage that can be expected to occur.
Maximum Pumping: Maximum canal or pool stage that can be pumped to, pump is subsequently turned off until stage decreases.
Normal Pumping: Expected normal pool elevations for impoundments and design tailwater stages for conveyance canal pump stations (flood damage reduction drainage discharge).
Minimum Pumping: Lowest canal or pool stage expected when pump may be turned on.
Minimum Non-Pumping: Lowest canal or pool stage that can be expected to occur under non-pumping conditions. In most cases, Minimum Pumping and Minimum Non-Pumping elevations are identical.

A.7.3.3.2.2 Levees

The proposed plan includes a new levee, L-67D (Blue Shanty Levee) to be constructed between the L-29 and L-67A Canals to provide a boundary for the Blue Shanty Flowway. The current L-67D freeboard is designed at 2.24 ft, as defined by vertical height between WCA 3B maximum historical recorded stage (9.76 ft NGVD) and embankment crest of 12.00 ft NGVD. Approximately 300 feet of the southern extent of the L-67D Levee will be constructed to the same crest elevation as the existing L-29 Levee to prevent rounding should a breach occur in the southern portion of the levee.

A.7.3.3.2.3 Existing Structures

<u>S-333 Gated Spillway</u>

S-333 is an existing gated spillway that is primarily used to make water deliveries from WCA-3A to Northeast Shark River Slough, with additional capability to deliver to the South Dade Conveyance System (SDCS) when required by the WCA-3A Regulation Schedule. The current capacity of S-333 is

1,350 cfs. The structure will be used with the proposed S-333N structure to increase the total conveyance capacity from southern WCA-3A to the L-29 Canal and ENP to a total of 2,500 cfs.

S-356 Temporary Pump Station

The S-356 structure is an existing temporary pump station located on the L-29 Canal, approximately one mile west of Krome Avenue. The current capacity of the structure is 575 cfs. As part of the CEPP project components, the current temporary S-356 pump station will be replaced with a permanent 1,000 cfs structure (1,650 cfs with redundant pumps) to contribute to project objectives. Should design components change for the permanent structure, further hydraulic analysis will be conducted in PED phase.

A.7.3.3.2.4 Existing Canals

L-29 Canal

The L-29 Canal delivers water from WCA-3A to the ENP Northeast Shark River Slough, where water is conveyed from the canal through a series of existing culverts and future bridges south into the ENP.

A.7.3.4 Risk and Uncertainty

This section presents qualitatively the risk and uncertainty associated with the project as designed for this PIR. Understanding that the current USACE philosophical approach to Feasibility Studies is to be quick and limit analyses to that for benefit and cost determinations, acknowledging risk and uncertainty in the hydraulic design of the project will be an important part of the risk registry. The overall approach to the hydraulic design was to be conservative enough to capture expected costs without being unrealistic in overestimation, yet not to underestimate beyond what optimization and the savings that could be realized during PED phase efforts.

A.7.3.4.1 Hydrologic and Hydraulic Computer Software Tools

Several hydrologic and hydraulic computer software tools were utilized in the formulation of alternatives and the Recommended Plan. Interpretation of hydraulic design results should consider the inherent strengths and limitations of the underlying hydrologic and hydraulic tools. Additional descriptions of the modeling tools are provided in Appendix A, Section A.8.1 (Modeling Strategy).

A.7.3.4.2 L-67D Blue Shanty Levee Feature

The L-67D Levee, known as the Blue Shanty Levee, is a true levee with a non-impounding flowway. The L-67D Levee height was initially estimated at six feet based on input from the project delivery team (PDT). The final levee height and footprint will be based on more detailed future analyses during PED that agrees with both agencies requirements, even though variances to typical requirements may be submitted with reasons based on PED analyses. Approximately 300 feet of the southern extent of the L-67D Levee will be constructed to the same crest elevation as the existing L-29 Levee to prevent rounding should a breach occur in the southern portion of the levee.

Potential Hazard

The L-67D Levee does not provide the function of a water impoundment barrier, but as a levee forming a boundary of a flowway that opens up into a vast floodplain (ENP). The potential risk associated with the levee, should the low probability of failure occur, remains low for the following reasons:

(1) The stage differences between WCA-3A, the flowway formed by L-67D, and eastern WCA-3B are low in magnitude, typically only a half foot to one foot between the flowway and WCA-3B under peak stage conditions. This stage relationship between the compartments is reflected today with the area between L-67A and L-67C, known as the “pocket”, that functions as a seepage step-down to prevent seepage out of WCA-3A from raising stages in eastern WCA-3B to the point of increasing flood risk to lands east of the C&SF East Coast Protective Levee System. Therefore, from an operational stage perspective (not volume rate of flow), the L-67D Levee will replace the function of the L-67C in the area of the flowway, thus not adding any additional risk to the C&SF system nor private lands that are within influence of water levels in the system.

(2) The new volume rate of flow is not expected to increase any potential of risk to the C&SF system nor private lands as it will flow unimpeded into a vast floodplain (ENP), which is the desired target for the additional water.

Freeboard under Design Conditions

The current Blue Shanty Levee freeboard as designed is 2.24 ft as defined by vertical height between the WCA-3B maximum historical recorded stage of 9.76 ft NGVD and embankment crest elevation of 12.00 ft NGVD. The embankment crest height was determined based on evaluating the maximum historical recorded stage at Site 71 in WCA-3B (9.76 ft NGVD), the maximum period of record stage modeled in the Blue Shanty Flowway (9.70ft NGVD for Alt 4R2), and applying an assumed Standard Project Flood (SPF) depth. The chosen maximum stage used for preliminary design was 9.7 ft NGVD, which is consistent with the Alt 4R2 operational stage constraint used for the L-29 Canal and for the S-632/S-633 inflows to the Blue Shanty Flow-way, with an assumed SPF depth of 17.5 inches (1.46 feet). The crest elevation was then rounded to 12.0 ft NGVD for conservative cost purposes.

Other Considerations in Determining Crest Elevation

(1) The main objective and purpose of CEPP is the restoration of the Everglades. One of the design goals is to minimize adverse impacts on the wetlands by providing the minimum levee footprint that provides an acceptable risk of conditions that may breach the levee.

(2) Stages in WCA-3A would be expected to top the Blue Shanty Levee crest, however, the L-67A Levee and structures will provide control over all inflows with the exception of seepage and direct rainfall.

(3) There will be little head differential between the area enclosed by Blue Shanty Levee and WCA-3B. In times of high stages within WCA-3A, seepage is of a magnitude that there always remains a small head differential (less than 3ft) between the two WCAs, and would be the same with the newly enclosed area. Additionally, since there would be a water depth inundating the toe, there would be no change in slope or a slope break, which is predominantly where headcutting actions begin that often lead to structure failure.

(4) As shown with the FEB wind-wave presentation, wind and waves could contribute to overtopping under combined extreme precipitation events. However, as stated previously, there are no lives at risk and no privately owned properties that may be damaged or at risk, should a breach occur. The area enclosed between L-67A and the proposed levee, as well as WCA-3B, is comprised of emergent marsh-type vegetation, especially near the canals (e.g. L-67A canal) where cattails are mainly found where phosphorous continues to remain high in the water column. Because of the vegetation, wind setup and wave generation would be minimal, e.g. FEB wind-wave effects presented in preceding section. With a shorter effective fetch than the FEB, the wind setup would be smaller and wave generation negligible.

L-67D Blue Shanty Levee Conclusion

Upon consideration of the following: (1) goal of minimizing impact with footprint, (2) Low HPC, (3) ability to endure extreme events, but perhaps not the superlative event such as a PMP or 100-yr stage with Category 5 Hurricane, (4) small head differential offering little to overtopping erosive action, and (5) that the Blue Shanty Levee poses no additional risk to the system, thereby offering no additional risk to the public, the PDT believes that the levee crest elevation is appropriate and within acknowledged acceptable risk.

A.7.3.4.3 Hydrologic and Hydraulic Lowering Risk Design

Gated Culverts in L-67A Levee

The USACE Engineering team utilized earlier culvert designs from the Modified Water Deliveries to Everglades National Park (MWD) project (Conveyance and Seepage component) that have been previously QA/QC. The previously designed structures have a larger capacity, 750 cfs versus CEPP capacity of 500 cfs. Therefore, risk to project function has been minimized. These gated culverts will be optimized during PED phase.

Seepage from WCA-3B to East

Seepage to the east from WCA-3B is one of the larger uncertainties. CERP, CEPP and all modeling before and in-between show that this is a major concern with proposed seepage management projects along L-30 (north of US-41) and L-31N (south of US-41). Too much seepage to the east toward private lands produces an adverse impact on flood damage reduction and thereby violating the Savings Clause in WRDA 2000. Many alternatives have been reviewed, all expensive, e.g. sheet pile. One alternative that has been implemented at a large scale is the use of “step-down” buffer areas (C-111 South-Dade project). Still, the expense and limited land availability makes complete implementation of such a system impractical for CEPP. Until seepage management is fully implemented, higher wet seasonal stages in WCA-3B will have to be constrained within tolerance of this known adverse impact. To ensure that seepage impacts to the east remain unchanged, CEPP proposed to increase the pumping capacity of the temporary 500 cfs S-356 Pumping Station to a permanent 1,000 cfs Pump Station. This pump station will take excess seepage collected in the L-30 and L-31N borrow canals and pump to the west so it may flow under US-41 into the ENP.

L-29 Removal with Bridge

The L-29 Levee is a necessary feature along the WCA-3B southern perimeter to prevent higher stages from impacting the US-41 (Tamiami Trail) highway. High stages in the L-29 borrow canal can impact the road sub-bed and cause the roadway to undulate. Prior to completion of the MWD Tamiami Trail Modifications project, there was limited conveyance under US-41 through the existing culverts, mostly because of limited head (i.e. L-29 borrow canal stage), sizing and minimum discharge due to vegetation and near topographical gradient. To improve conveyance and natural habitat connectivity, bridge(s) and associated Tamiami Trail roadway modifications have been proposed as part of both the MWD project and the DOI Tamiami Trail Next Steps (TTNS) project. Construction of the MWD project 1-mile eastern bridge and roadway modifications to accommodate L-29 Canal stages up to 8.5 feet NGVD was recently completed in December 2013.The TTNS project will accommodate L-29 Canal stages up to 9.7 feet NGVD.

If the MWD and TTNS bridges and the associated Tamiami Trail roadway modifications are constructed and completed as expected, then the L-29 Levee can be degraded with minimum, but acceptable, risk to US-41. However, if these projects are not constructed, then L-29 Levee degrading is at risk because of potential damage to US-41. If the L-29 Levee cannot be degraded, then stages within the L-29 borrow canal will remain controlled with a maximum operating stage limit between 7.5 and 8.5 feet NGVD (operational constraints will be determined with the MWD Final Operating Plan) to minimize potential damage to US-41.

This also impacts the operation of the flowway created with construction of the Blue Shanty Levee feature. Since the Blue Shanty Levee is a levee and is not constructed to impound water, the benefits to be gained with flowing of water through the area enclosed by L-67A and the Blue Shanty Levee may not be realized until the TTNS bridges and roadway modifications are constructed.

A.7.3.5 Hydraulic Design Data Sheets

TABLE A-30. S-631, S-632, S-633 GATED CULVERTS

HYDRAULIC DESIGN DATA SHEET

Location Along southern portion of L-67A
S-631: x = 766,105 y = 543,464
S-632: x = 771,712 y = 543,479
S-633: x = 788,604 y = 568,318

Purpose S-631 conveys flows from the WCA-3A to through the Blue Shanty Flowway to provide flows to the Shark River Slough.

Design Conditions	Discharge	500	cfs
	Headwater Elevation	8.60	feet, NGVD 29
	Tailwater Elevation	8.10	feet, NGVD 29
Culvert Data	Number of Barrels	1	
	Barrel Type	Concrete Box Culvert	
	Box Width	11.0	feet
	Box Height	11.0	feet
	Culvert Length	100.0	feet
	Upstream Invert	-4.50	feet, NGVD 29
	Downstream Invert	-4.50	feet, NGVD 29
	Natural Grade	9.00	feet, NGVD 29
	Natural Water Table	9.00	feet, NGVD 29
	Headwall - HW Elevation	TBD	feet, NGVD 29
	Headwall - TW Elevation	TBD	feet, NGVD 29
	Wingwall - HW Elevation	TBD	feet, NGVD 29
	Wingwall - TW Elevation	TBD	feet, NGVD 29
Canal Data	Side Slopes (V:H)	1 on 2	
	Upstream Bottom Width	40.0	feet
	Upstream Bottom Elevation	1.40	feet, NGVD 29
	Downstream Bottom Width	40.0	feet
	Downstream Bottom Elevation	1.40	feet, NGVD 29
Energy Dissipation	**Riprap Requirements**		
	Rip Rap Design Velocity	4.13	fps
	Upstream Length	TBD	feet
	Upstream Protection Elevation	TBD	feet, NGVD 29
	Downstream Length	TBD	feet
	Downstream Protection Elevation	TBD	feet, NGVD 29

TABLE A-31. S-333N GATED SPILLWAY
HYDRAULIC DESIGN DATA SHEET

Location L-67 Canal south of the L-67A and L-67C intersection; XY coordinates TBD

Purpose Works in conjunction with S-333 to increase flow capacity (total of 2500 cfs) from WCA 3A to Northeast Shark River Slough.

Design Conditions	Discharge	1150	cfs
	Headwater Elevation	7.50	feet, NGVD 29
	Tailwater Elevation	7.00	feet, NGVD 29
Crest Data	Shape	Ogee	
	Design Head (Hd)	10.6	feet
	Net Crest Length	29.0	feet
	Crest Elevation	-3.10	feet, NGVD 29
	Approach Apron Elevation	-6.00	feet, NGVD 29
	Weir Control	Vertical Slide	
Gates	Number of Gates	1	
	Gate Width	29.0	feet
	Gate Height	14.6	feet
	Clearance Elevation	15.60	feet, NGVD 29
	Breastwall Elevation	TBD	feet, NGVD 29
	Intermediate Pier Width	3.25	feet
Stilling Basin	Design Discharge	1,150	cfs
	Apron Elevation	-6.00	feet, NGVD 29
	Apron Length/Width	39.0/29.0	
	End Sill Elevation	-5.50	feet, NGVD 29
	Top of Baffle Block Elevation	-5.00	feet, NGVD 29
	Dist from crest toe to 1st row of blocks/2nd row	19.50	feet
	Velocity over End Sill	3.17	fps
	Training Wall Elevation	TBD	feet, NGVD 29
Canal Data (US/DS)	Invert - Thalweg	-10.0	feet, NGVD 29
	Top of Bank	15	feet, NGVD 29
	Bottom Width	35.0	feet
	Top Width	135.0	feet
	Side Slope (V:H)	1 on 2	
Revetment	Riprap Extent (Downstream)	TBD	feet
	Riprap Size (D50)	TBD	feet
	Riprap Specific Weight	TBD	lb/ft3
	Max Velocity Riprap Can Withstand	TBD	fps

TABLE A-32. S-355W GATED SPILLWAY
HYDRAULIC DESIGN DATA SHEET

Location L-29 canal, adjacent to the proposed L-67D Levee; x = 785,915 y = 519,337

Purpose Convey flows from Blue Shanty Flowway eastward to help meet ecological objectives.

Design Conditions	Discharge	1,230	cfs
	Headwater Elevation	9.70	feet, NGVD 29
	Tailwater Elevation	8.70	feet, NGVD 29
Crest Data	Shape	Ogee	
	Design Head (Hd)	5.7	feet
	Net Crest Length	36.0	feet
	Crest Elevation	4.00	feet, NGVD 29
	Approach Apron Elevation	-4.00	feet, NGVD 29
	Weir Control	Vertical Slide	
Gates	Number of Gates	3	
	Gate Width	12.0	feet
	Gate Height	8.0	feet
	Clearance Elevation	9.00	feet, NGVD 29
	Breastwall Elevation	TBD	feet, NGVD 29
	Intermediate Pier Width	3.25	feet
Stilling Basin	Design Discharge	1,230	cfs
	Apron Elevation	-4.00	feet, NGVD 29
	Apron Length/Width	36.0/42.5	
	End Sill Elevation	-3.50	feet, NGVD 29
	Top of Baffle Block Elevation	-2.00	feet, NGVD 29
	Dist from crest toe to 1st row of blocks/2nd row	18.00	feet
	Velocity over End Sill	2.37	fps
	Training Wall Elevation	TBD	feet, NGVD 29
Canal Data (US/DS)	Invert - Thalweg	-7.6	feet, NGVD 29
	Top of Bank	15	feet, NGVD 29
	Bottom Width	50.0	feet
	Top Width	140.0	feet
	Side Slope (V:H)	1 on 2	
Revetment	Riprap Extent (Downstream)	TBD	feet
	Riprap Size (D50)	TBD	feet
	Riprap Specific Weight	TBD	lb/ft3
	Max Velocity Riprap Can Withstand	TBD	fps

TABLE A-33. S-356 PUMP STATION
HYDRAULIC DESIGN DATA SHEET

Location	L-29 canal; vicinity of the existing temporary S-356 pump station; x = 820,014 y = 519,597			
Purpose/Operational Intent:	Seepage Control and Water Supply S-356 Pump station will replace the existing temporary S-356 pump to provide permanent seepage return to Northeast Shark River Slough.			
Design Condition:			1,000	cfs
Design Capacity :			**1,650**	**cfs**
Pump Station Capacity Criteria:				
	Required estimated seepage collection rate			
Number of Pumps			4	
Pump Mix Type and Size		Electric	(1) 150	cfs
		Diesel	(3) 500	cfs
Mix Criteria:				
	1. The pump station will have 4 bays; one 150 cfs electric motor, and three 500 cfs diesel engines.			
	2. The pump mix allows for a range of seepage rates, while having duplicate pumps for operation and maintenance consideration			
Control			TBD	
Design Heads				
	Normal		TBD	ft, NGVD
	Maximum (HW=5.5 NGVD, TW = 8.5 NGVD)		3.00	ft, NGVD
Intake Water Surface Elevations				
	Maximum Non-Pumping Pumping		TBD	ft, NGVD
	Maximum Pumping		TBD	ft, NGVD
	Start Pumping		TBD	ft, NGVD
	Normal Drawdown		TBD	ft, NGVD
	Minimum Drawdown Pumping		TBD	ft, NGVD
	Minimum Non-Pumping		TBD	ft, NGVD
	Channel Invert		TBD	ft, NGVD
Discharge Water Surface Elevations				
	Maximum Non-Pumping		TBD	ft, NGVD
	Maximum Pumping		TBD	ft, NGVD
	Normal Pumping		TBD	ft, NGVD
	Minimum Pumping		TBD	ft, NGVD
	Minimum Non-Pumping		TBD	ft, NGVD
	Channel Invert		TBD	ft, NGVD

A.7.4 STRUCTURAL DESIGN

A.7.4.1 General Status of Completed and Non-Executed Efforts

Structural design of S-631, S-632, S-633, S-333N, S-355W and S-356 will be completed during the design phase. During design phase the structural calculation will be completed after survey, hydraulic design, and geotechnical investigations are performed. The structural design will conform with the appropriate Engineering Manuals (EM), Engineering Regulations (ER), or Design Criteria Memorandums (DCM).

A.7.4.2 Pumping Stations

S-356 is a seepage pump station that will be similar in design to Miller (S-486)/Merritt (S-488) pump stations.

A.7.4.3 Overflow Spillways

S-333N and S-355W are gated structures similar to S-65EX1, using a two-phased approach and offsetting the structures will not require a bypass canal to be designed for construction of the structures.

A.7.4.4 Culverts

S-631, S-632, and S-633 are gated culverts that will be designed similar to the culverts on Decomp. Since the culverts have to be placed submerged, during design phase these may be changed.

A.7.5 MECHANICAL AND ELECTRICAL DESIGN

A.7.5.1 General

The pumping station mechanical design shall be in accordance with Hydraulic Institute Standards, EM 1110-2-3102 (General Principles of Pumping Station Design and Layout), and EM 1110-2-3105 (Mechanical and Electrical Design of Pumping Stations). The design will also follow the guidance of ETL 1110-2-313 (Hydraulic Design Guidance for Rectangular Sumps of Small Pumping Stations with Vertical Pumps and Ponded Approaches).

The existing Pumping Station 356 will be removed and a new, larger pumping station will replace it. The new station will have a required pumping capacity of 1,000 cfs.

The pump mix will be further developed during the design phase of the project, but it will likely have a mix similar to having three 500-cfs diesel engine driven pumps and one 150-cfs electric motor-driven pump.

The pump intakes will likely be suction bell type. The use of formed suction intakes at the pumps shall be evaluated during preparation of the plans and specifications for the pumping station and shall be based upon the channel intake design.

Axial flow pumps will be used for the pumping station. The decision on whether the pumps will have either a conventional or siphon discharge will be determined during the preparation of the plans and specifications.

The pumping station electrical design shall be in accordance with NEC, NFPA, IESNA, TIA/IEA, IEEE, and recommended practice. Also, EM 1110-2-3102 (General Principles of Pumping Station Design and Layout) and EM 1110-2-3105 (Mechanical and Electrical Design of Pumping Stations) will be used.

Although the capacity of this station is low enough that SFWMD's Major Pumping Station Engineering Guidelines is not applicable, we will follow the applicable portions of these guidelines.

The pumping station will be similar in design to two of the stations for the Picayune Strand Project, Merritt Pumping Station (S-488) and Miller Pumping Station (S-486). Information on these two stations is provided in **Table A-34**. Plates M-1 through M-4, in Appendix A, Annex D-1, show plan and section views of Merritt Pumping Station, along with a plan view of its diesel fuel system.

TABLE A-34. PICAYUNE STRAND PUMPING STATIONS

Picayune Strand Pumping Stations				
Name	Capacity cfs	Diesel Engine Driven Pumps	Electric Motor Driven Pumps	# of Bays
Miller (S-486)	1,560	6 – 235 cfs	2 – 75 cfs	8
Merritt (S-488)	1,030	4 – 220 cfs	2 – 75 cfs	6

A.7.5.2 General Status of Completed and Non-Executed Efforts

Mechanical and electrical design of S-631, S-632, S-633, S-333N, S-355W and S-356 will be completed during the design phase. During design phase the mechanical and electrical calculations will be completed after the hydraulic design is performed, the pump mix is determined, and the structures' normal and emergency operating parameters are finalized.

A.7.5.3 Pumping Station S-356 Replacement Features

The larger pumps may be designed with a Formed Suction Intake (FSI) and have rectangular pipe for the intake and discharge. These pumps may be conventional-discharge or siphon-discharge type. The pumps will be driven by diesel engines through right-angle reduction gears. One of the pumping systems for this station is a redundant system as required by SFWMD's Major Pumping Station Engineering Guidelines.

The smaller pump is intended for seepage control. Its flow rate is not included in the total flow capacity of the station. The pump will be an axial-flow-type vertical-shaft pump. The pump will be driven by a direct-drive electric motor.

The diesel engine driven pumps are expected to run at less than 500 rpm with an efficiency of about 80%. The diesel engine pump drives for the 500-cfs pumps should be about 1,200 horsepower each.

The pumping station will include various support items, including the following:

a. Diesel fuel system, including vaulted double-wall aboveground fuel storage tanks capable of holding enough fuel to operate two of the engine driven pumps and an emergency generator continuously for seven days.
b. Hoisting system for maintenance or repair of the pumping equipment.
c. Toilet facility with a water closet and a lavatory.
d. Kitchen-type sink.
e. Potable water system and a septic system for the plumbing fixtures.
f. Ventilation system to provide fresh air in the pump bays, generator area, and toilet room.
g. Air-conditioning system for the office.
h. Stilling wells containing float switches to be used for pump operations and water level monitoring.

A.7.5.3.1 Pumping Station Features

Pump Drives
The diesel engines will be standard model full-diesel types, 2 or 4 cycle, with mechanical injection and cooling provided by keel coolers. Diesel engine horsepower will be about 1,200 hp.

Engine Auxiliary System
Cooling of each main engine will be by means of a closed system consisting of keel coolers, overhead expansion tanks, and engine-driven jacket water and aftercooler water circulating pumps with proper heat balance maintained by the thermostatically controlled proportioning valves. The main lubricating oil pump for each main engine will be driven directly by the engine.

Speed Reduction Gears
Power will be transmitted from the engines to the pumps by means of right-angle type gear reducer units. These units will be designed for an application factor of 2.0 times the maximum input power. Thrust load due to hydraulic unbalance and an anti-friction type bearing located within the reducer unit will carry the weight of pump rotating elements. Connection between reducer and engine will be by flexible coupling to compensate for misalignment and vibration or shock transmission. The reducer will be provided with forced lubrication from a direct connected positive displacement pump with cooling of the oil by an external system. To prevent reverse rotation, the transmission would be fitted with an anti-reverse rotation clutch.

Fuel Oil Storage System and Supply
Aboveground Storage Tanks (ASTs) will be located at a safe distance from the station. ASTs shall be concrete-vaulted and have a dual containment feature. Fuel capacity may be as much as 36,000 gallons and several tanks may share this capacity. Fuel capacity should be for seven days, 24-hour/day continuous operation at maximum fuel consumption rate. The tanks will be filled from a fuel truck. The tanks will be connected to the station supply header. The fuel system for each main engine will consist of a day tank (typically up to 250 gallons capacity) to supply the diesel engine. The day tank will have automatic operation in sending and receiving fuel and controlling the level of the fuel inside of the day tank. A similar day tank will be provided for the engine generator sets.

Station Crane/Hoist
An overhead bridge-type electric crane will be provided. The crane/hoist shall be capable of handling up to 30-ton loads. The crane/hoist will handle pumping station equipment, such as diesel engines, reduction gears, or pump components during initial installation, as well as for general service thereafter.

Diesel Engine-Generator Sets
Two diesel engine-driven generator sets with capacities up to 500 kW each may be provided. These generators must provide sufficient power to operate the station at full capacity, including running all auxiliary equipment continuously for as long as seven days. One genset may also provide general standby power.

Potable Water and Plumbing
A potable water supply and plumbing system will be provided. This will include a septic system. A filtered water system will be necessary for the station to supply water to a Toilet (lavatory, shower, and water closet) and small kitchen area.

Air Conditioning
Small split-system air conditioning systems will be provided for the control room, telecommunications room, and the break room.

Ventilation System
A system of air inlet openings and exhaust fans will be provided for ventilation of the operating floor area. The air inlet louvers will be the type commonly referred to as Miami-Dade louvers. Bird screening will also be provided over the openings. The wall type exhaust fans will have motor-operated dampers.

Trash Rake
Trash rake/rack system will be one of two types: an automatic, continuously rolling, flex rake and trash rack system such as that manufactured by Duperon, or a powered rail-mounted traveling trash rake and hoist car assembly with a telescoping arm used to grip and remove debris. This system is similar to ones that are manufactured by Hydro Component Systems. The system selected shall be similar to those that have proven satisfactory at previously completed pumping stations.

Pump Model Tests
The specifications will require that a series of model tests be performed to verify performance and cavitation limits of the proposed pump. The contractor will be required to construct one complete pumping system for each size pump to the necessary scale model. The pumping system will include the forebay, pump, and discharge tube. All tests for determination of compliance with guarantees of capacity and/or efficiency will be accomplished using prototype heads.

A.7.5.3.2 Electrical Features

Electric Service and Backup Generator
A 480-volt, three phase, electrical service shall be provided using the existing power lines that run alongside Tamiami Trail. Transient Voltage Surge Suppression (TVSS) shall be provided at the service entrance. The local utility company shall provide the power. Diesel engine-generator units shall be provided to supply 480-volt, three phase electrical power when utility power is not available or not reliable. Backup generators and automatic transfer switches shall be sized sufficiently to power diesel engine auxiliaries, trash rakes, exhaust fans, lights and SCADA equipment.

Interior Electrical Distribution
Switchgear rated for 480 volt, three phase with a main breaker will be connected to the incoming service and will feed engine control centers, motor control centers, lighting panels, power panels and station equipment defined in the Pumping Station Features above. Each engine control center will house starters and controls for auxiliary equipment for each respective engine unit. Main switchboard will also feed transformers to supply 120/208 or 480/277 volt loads as necessary.

Interior and Exterior Lighting
High intensity discharge, industrial high bay luminaries will be used for the main pumping station area with industrial fluorescent fixtures with electronic ballasts for office and general type areas. Exterior lighting for security purposes would be automatically controlled by photo-electric cells and contactors.

Wiring and Conduit
Insulated copper conductors will generally be installed in either PVC coated rigid galvanized steel conduit or schedule 80 rigid plastic conduit. Conductors will be rated for 600 volt insulated types XHHW or XHHW-2. All wiring will conform to UFGS Guide Specifications.

Instrumentation and Controls
The pumping station will have a centralized monitoring and control room. Each diesel engine pump drive will have a separate motor control center to supply power and house controls for engine auxiliaries, such as jacket water pump, engine lube pump, fuel filter pump, etc. Diesel engines will also be equipped with a separate instrument panel and will house engine start/stop controls and pressure and temperature indicators to indicate engine performance. Programmable logic controllers (PLCs) will be used to monitor and control the engines and station auxiliaries. An Ethernet network will connect the PLCs and station computer. Ethernet based IP cameras will also connect to the Ethernet network. The station computer will allow for operation of the station via SFWMD's preferred SCADA software.

SCADA and Telemetry
The controls systems shall include manual, automatic and telemetry capabilities for the pumps and auxiliary systems. The engine start/stop controls shall operate locally at the engine, remotely from the control room, and from the central control station. The automation components of all pumping stations and structures that will eventually be operated and maintained by South Florida Water Management District (SFWMD) must conform to SFWMD standards in order to (1) achieve cost efficiency in design, construction, and operation and maintenance, (2) meet safety, reliability, and performance requirements during routine and emergency operations. The automation components are broadly defined to include hardware, software, communications, and user interface elements.

A.7.5.4 Gated Spillways

Gate Operators
Gate operators will be designed based on the size, weight, and hydraulic loading on the gates. The operators will either be electric motor driven through a drum and pulley system or via as an actuator on a stem screw.

Electrical Service
A control center will house a main breaker, combination starter for the gate motor, lighting panel, relay compartment, and a circuit for exterior lighting. Surge suppression will be provided for each electrical/electronic system within or outside the structure.

Control and Monitoring
Duplicate open-close push button station in the control house and at the spillway structure will be provided for manual gate control. Necessary open, close, automatic control relays, and limit switches will be incorporated in the gate control circuit. Power and control circuits for water level recorders and gate position recorders will be provided.

A.7.5.5 Telemetry

Each spillway site that requires remote automation will be equipped with an RTU compatible with the existing SFWMD telemetry system. RTU software will be in accordance with the SFWMD standard load set. The construction plans will contain plans for a fully functioning telemetry system capable of connecting to and communicating with the SFWMD existing system. Additional coordination during the development of plans and specifications will finalize the telemetry requirements.

A.8 HYDROLOGIC MODELING

A.8.1 Modeling Strategy and Tools

The primary application of models in the CEPP is for the assessment of regional-level hydrologic planning. More detailed models were also applied to address specific questions related to hydraulic and water quality constraints. The CEPP modeling tools were jointly selected by the USACE Jacksonville District (SAJ) and the South Florida Water Management District (SFWMD) in October-November 2011 based on their collective capability to provide adequate hydrologic information to conduct evaluations of the entire south Florida system for the needs of the CEPP. Due to the time required to complete prerequisite model documentation, documentation review, and compilation of this model validation review package, the expedited CEPP schedule did not afford the opportunity to submit the proposed modeling tools for USACE Engineering software validation evaluation prior to execution of the modeling strategy and application of the initial recommended modeling tool suite, which initiated in January 2012. However, prior to implementation of the CEPP modeling, the CEPP modeling strategy was vetted through USACE at the SAJ District, South Atlantic Division (SAD), and Headquarters (HQ) levels through the prior CEPP periodic in-progress reviews (IPR-1 in December 2010; IPR-2 in January 2012) and CEPP Decision Point 1 vertical coordination meeting (January 2012). Prior to completion of the hydrologic modeling of the CEPP final array of alternatives, all CEPP modeling tools were reviewed and approved for use through either the USACE Engineering software validation process or through the CEPP Agency Technical Review (ATR) process, as further documented in Section A.8.1.1.

The CEPP modeling strategy centered around use of a decoupled link-node model Regional Simulation Model for Basins (RSM-BN) for the EAA, Stormwater Treatment Areas (STAs) and the northern estuaries, in combination with a detailed meshed Regional Simulation Model for the Glades and Lower East Coast Service Areas (RSM-GL) for the Water Conservation Areas (WCAs), Everglades National Park (ENP) and the Lower East Coast (refer to section A.8.1.2 for additional documentation of the RSM models). The CEPP modeling strategy provides an overview of the modeling tools, including maps of the model domains, applied throughout the plan formulation process and how the tools were applied in support of the CEPP planning process (refer to Reference 1, included with the Hydrologic Modeling Annex A-2).

Preliminary screening assessments for Lake Okeechobee, the northern estuaries, and the impoundment storage within the EAA, collectively referred to as the "North of the Red Line components," utilized the Reservoir Sizing and Operations Screening (RESOPS) model, the Lake Okeechobee Operations Screening (LOOPS) model, and the C-43 Spreadsheet Model. Preliminary screening assessments for the Water Conservation Areas (WCAs) and Everglades National Park (ENP), collectively referred to as the "South of the Red Line components" (including the components at the EAA/WCA Red Line boundary, in addition to the Green/Blue/Yellow Line components) utilized the iModel optimization tool and limited-scope sensitivity simulations using the RSM-GL. For the final array of alternatives, analysis of the North of Red Line components and the South of the Red Line components were conducted using the RSM-BN and the RSM-GL, respectively. The RSM-GL model was also used for performance evaluation within the Lower East Coast Service Areas, areas which were not encompassed within the domain of the iModel used during preliminary screening. The Hydrologic Engineering Centers' River Analysis System (HEC-RAS) modeling tools were utilized for hydraulic design efforts to evaluate potential canal conveyance modifications and structural modifications identified with the CEPP Recommended Plan components. The Dynamic Model for Stormwater Treatment Areas, Version 2 (DMSTA2) was utilized during preliminary screening and final array modeling to confirm compliance with required State of Florida water quality standards.

Additional technical descriptions for the RSM-BN, RSM-GL, and HEC-RAS hydrologic and hydraulic modeling tools applied to evaluation of the CEPP final array of alternatives are provided in Sections A.8.1.2 through A.8.1.4. From initial formulation through selection of the Recommended Plan (Alternative 4R2), the CEPP modeling strategy has not included the application of detailed flood event modeling (or hydrodynamic levee assessment). It is expected that higher resolution hydrologic and hydraulic modeling tools will be required to further analyze localized and possibly regional-scale effects of specific components of the CEPP Recommended Plan, with the scope of these analyses further identified during the Pre-Construction Engineering and Design (PED) phase of the project.

A.8.1.1 Overview of USACE Model Validation Process and CEPP Approach

The USACE model certification process distinguishes between "Engineering" and "Planning" models. One of the goals of the Scientific and Engineering Technology (SET) initiative is to inventory and evaluate the software used by the Corps' scientific and engineering community, to ultimately achieve a manageable and cost-effective USACE corporate tool set. Each piece of software is inventoried, reviewed, and ultimately listed in one of five categories: Enterprise Tools, Community of Practice (CoP) Preferred, Allowed for Use, Retired, and Not Allowed for Use. It is expected that the lists will continue to evolve as new software is introduced.

Current USACE guidance (ES-0801: June 2011) regarding software validation for the Hydrology, Hydraulics, and Coastal Community of Practice (HH&C CoP) indicate that both the District and Division need to recommend the software for evaluation. The recommendation should state whether the software will be used nationally, regionally, or locally, and should include why the software is needed, an explanation as to what it does and how it does it, why any of the other corporate software already on the list doesn't meet the needs, who within the Corps has knowledge of this software, what type of peer review has it received, what Area of Expertise (AoE) software list should it be included with, and what documentation, training and support can be found. The goal of the SET program is to manage the number of pieces of software so the Corps doesn't have to support multiple pieces of software that do roughly the same thing. The USACE should use "well-known and proven" software unless a new piece of software does something one of the "validated" pieces of software does not.

Based on ES-0801 guidance, the following general information (items 1 through 9) is typically requested to support a request for USACE "Engineering" model validation evaluation.

- Model Classification (Area of Expertise);
- Requested Model Application Area;
- General model documentation/description of model capabilities (include web site links or documentation reports);
- Why the model software is needed? (consider other approved corporate software);
- External peer review (requested by?; Conducted by?; Model version and date?; final reports should be provided);
- Internal technical review by Interagency Modeling Center (model version and date?; final report should be provided);
- Previous applications of the model (specific projects and sponsor agency);
- Additional applicable reports or documentation, if any (other agency peer reviews, project specific applications of model, model users' guide, etc.);
- USACE knowledge base for this software.

The RSM-BN, RSM-GL, and DMSTA models were reviewed through the HH&C CoP validation process for engineering software, as part of the CEPP project. The RSM (including RSM-BN and RSM-GL) and DMSTA models were both classified as “allowed for use” for South Florida applications in August 2012 and January 2013, respectively. The Hydrologic Engineering Centers’ River Analysis System (HEC-RAS), developed by the USACE HEC, has been previously reviewed and classified as a “CoP Preferred” hydraulic design and river hydraulics modeling tool.

ES-0801 also provides guidance regarding “model building pieces of software.” While they are used frequently in USACE planning and engineering processes they are pieces of software that can be used to create any type of model. Therefore, they are impossible to pre-certify through the standard engineering software validation process. However, the HH&C CoP believes these tools are fine for building models and thus they are "validated" as "Allowed for Use". Still, the HH&C CoP cautions the user that the Agency Technical Review (ATR) and the Independent External Peer Review (IEPR) must include a much more thorough review of the inner workings of the model, as the basic assumptions, equations and output used or created for the model have not been pre-validated. It is also important to note that an application built using these tools needs to be created by someone knowledgeable about the software and that experienced users must review the application during the review process, ATR and IEPR, if required, to ensure the validity of any equations and/or algorithms built into the tool. The Project Review Plan should reflect and detail technical requirements for individuals reviewing these tools.

For the CEPP, based on coordination with the USACE South Atlantic Division (SAD) and subsequent coordination with the CEPP USACE ATR team, it was determined that four model building software tools used during the initial CEPP screening process would be reviewed as part of the CEPP ATR: the RESOPS, LOOPS, and C-43 spreadsheet model tools; and the iModel optimization tool. ATR review and approval of these modeling tools for CEPP application was completed in November 2012. Additional descriptions of these modeling tools are provided in the CEPP modeling strategy (Reference 1, included with the Hydrologic Modeling Annex A-2).

A.8.1.2 Modeling Tool Overview: Regional Simulation Model (RSM-BN and RSM-GL)

South Florida is a unique environment requiring specialized models to simulate regional operations. South Florida has a complex regional hydrologic system that includes thousands of miles of primary and secondary networked canals, nearly 300 man-made flow-regulation structures, thousands of square miles of nearly flat terrain much of which are wetlands, and permeable surficial soils that enhance groundwater-surface water interactions. Hydrologic and hydraulic analyses of this complex system require a computational model that can run quickly, offer flexibility, and generate output that can be clearly interpreted. Because of the region's highly variable hydrology (extreme rain events and periods of extended droughts), it is imperative that models be capable of running regional simulations of decades covering wet, dry and average rainfall conditions. Finally, land use changes and water demands for this extended period of time requires the user to easily modify input data sets, as well as an ability to use generalized data sets to optimize performance.

The Regional Simulation Model (RSM) was developed by the South Florida Water Management District (SFWMD) to overcome these limitations, beginning in 1994. RSM provides the computational framework for developing more complete and numerically sound integrated surface water and groundwater models where both components receive equal attention. The RSM was developed to eventually replace the SFWMD South Florida Water Management Model (SFWMM) for simulating the water management in the Central and Southern Florida Flood Control Project (C&SF Project). The RSM simulates the hydrology and water management of the South Florida region, providing modeling support to regional restoration, flood control, and water supply planning efforts. The same RSM program executable is used by the link-node RSM-BN and the mesh-based RSM-GL (or any other mesh-based sub-regional RSM models). The RSM is an implicit, finite-volume, continuous, distributed, and integrated surface-water and ground-water model.

Development of a regional South Florida RSM (SFRSM) model, as originally envisioned, has not been completed at this time. Due to this limitation, the RSM currently is applied to sub-regions within the south Florida domain. Each of the sub-regional models was created to address specific water resource management issues or to support alternative plan formulations for the CERP. Prior to the CEPP, the RSM model has been utilized to develop sub-regional models to support modeling evaluations for both SFWMD projects and USACE/SFWMD CERP projects: CERP WCA-3A Decomp (USACE/SFWMD); CERP Biscayne Bay Coastal Wetlands (SFWMD); Lake Okeechobee Watershed Construction Project (SFWMD); and the Caloosahatchee River Watershed Protection Plan (SFWMD). The following RSM sub-regional model applications had been previously developed and applied: Biscayne Bay Coastal Wetlands/C-111 (refer to Figure A.8-1 for model domain), Glades and Lower East Coast Service Areas (Glades-LECSA, or RSM-GL; refer to Figure A.8-2 and Figure A.8-3), and Northern Everglades (NERSM; refer to Figure A.8-4 and Figure A.8-5) models.

A.8.1.2.1 Regional Simulation Model for Basins (RSM-BN)

Although RSM is principally applied for irregular triangular mesh models, RSM can be used as a node-link model when implemented in a study area that can be conceptualized as a lumped system, as in the case RSM-BN. The RSM-BN model domain covers Lake Okeechobee, the EAA, and three major watersheds: the Kissimmee, the St. Lucie River, and the Caloosahatchee River. The link-node based model is designed to simulate the transfer of water from a pre-defined set of watersheds, lakes, reservoirs or any "waterbody" that either receives or transmits water to another adjacent waterbody. The watersheds are further divided into sub-watersheds until fundamental waterbodies can be considered as separate model nodes. RSM produces complete water budgets given appropriate boundary conditions and simplified operating rules. The NERSM was the precursor model to the RSM-BN; the RSM-BN is NERSM with added explicit simulation of the EAA, which is a critical component for the CEPP as the proposed FEB is located within the EAA footprint. The node link representations of the EAA in the RSM-BN baseline conditions and the CEPP alternatives 1 through 4R2 are provided in Figures A.8-6 through A.8-7; Figures A.8-4 through A.8-8 collectively represent the model domain and node-link assumptions utilized for the RSM-BN CEPP application.

Prior to the CEPP, the RSM-BN model was used by the SFWMD to support the SFWMD River of Grass (ROG) planning effort (2008-2009) and the SFWMD northern Everglades planning process, and the model was well-received by the public stakeholders. Limitations of the RSM-BN model are consistent with limitations previously documented for the precursor NERSM:

- A formal model calibration was not conducted for the NERSM or RSM-BN models; the original NERSM application, for the SFWMD Lake Okeechobee Watershed Construction Project, conducted a simplified model validation through comparison of NERSM results with current base and future base simulations represented with the SFWMM and Upper Kissimmee Model (UKISS).
- More advanced capabilities of RSM such as 1-dimensional canal flow routing and 2-dimensional overland flow/groundwater flow calculations were not used in RSM-BN. Groundwater hydrology is not explicitly represented within the RSM-BN.
- Within an RSM node-link model, the following framework applies: each node represents a waterbody (hydrologic basin, lake, reservoir, STA, etc.); each link acts like a conduit only, with no hydrologic/hydraulic simulation for the links; and individual operating and management rules at each structure define the linkage of all nodes within the modeling domain (hydrologic processes are simulated for each node at varying complexities).
- For the CEPP application, the RSM-BN model domain includes a simplified model representation of the L-4, L-5, and L-6 Canals located along the CEPP red line interface; discharges from STA-2, STA-3/4, and STA-5/6 are routed directly to the downstream WCAs per CEPP operating protocols, where these WCA inflows are captured by the RSM-GL model.
- For the CEPP application, water is routed through storage features assuming a level pool with negligible slope in the water surface. The assumption is valid as long as the volume entering a storage feature during the 1-day time step is small relative to the volume of water in storage. Sloped water surfaces can be simulated as an option within the RSM-BN (for example, for representation of the Kissimmee River flood plain).
- The model simulates the management of the system according to a set of operational criteria referred to as management rules. These rules are expressed in regulation schedules, gate-operation criteria, and established rules governing the operation of the structures. It is assumed that the management rules prescribed for the various simulation scenarios are reasonable for the variety of hydrologic conditions represented by the period of simulation.
- A daily time step is assumed to be adequate for planning purposes and the evaluation of RSM-BN performance measures. Most measures are expressed in terms of annual, monthly, and weekly statistics.
- Historical flow patterns from the adjacent sub-watersheds contributing into Lake Okeechobee are assumed to be preserved while simulating management measures. Rainfall-runoff relationships and drainage/routing characteristics within a sub-watershed are assumed not to change from before to after management measures are operational.

- It is assumed that a change in management rules will not change the historical hydrologic variables.
- Other than the footprint associated with management measures considered in the future base and alternative scenarios, it is assumed that there are no other changes in land use or land cover within the RSM-BN for the CEPP application. Variable land uses can be assumed in the EAA and Kissimmee River watershed domains of the RSM-BN via model input pre-processing.
- The lower Kissimmee River and floodplain between consecutive water control structures is assumed to be hydrologically similar to a level-pool reservoir with a unique stage-volume relationship. Lock operations are not simulated.

A.8.1.2.2 Regional Simulation Model for Glades and Lower East Coast Service Areas (RSM-GL)

The RSM-GL model domain covers an area of 5,825 square miles and encompasses a total of thirteen basins (Figure A.8-2): 1) L-28 Interceptor; 2) L-28 Gap; 3) Feeder Canal; 4) East Collier; 5) Everglades National Park (ENP); 6) Water Conservation Area 1 (WCA-1) ; 7) WCA-2A; 8) WCA-2B; 9) WCA-3A; 10) WCA-3B; 11) Lower East Coast Service Area 1; 12) Lower East Coast Service Area 2;and, 13) Lower East Coast Service Area 3. The southern, eastern and southwestern boundaries of the model are comprised of Florida Bay, the Atlantic Ocean/Biscayne Bay and the Gulf of Mexico coastlines, respectively.

The RSM-GL can simulate one-dimensional canal/stream flow and two-dimensional overland and groundwater flow using a variable triangular mesh. The overland and groundwater flow components are fully coupled in the RSM and RSM-GL for a more realistic representation of runoff generation. The RSM-GL has physically-based formulations for the simulation of overland and groundwater flow, evapotranspiration, infiltration, levee seepage, and canal and structure flows. The model uses the diffusive wave approximation of Saint-Venant's equation to simulate canal and overland flows. This model is capable of simulating features that are unique to South Florida such as low-relief topography, high water tables, saturation-excess runoff, depth-dependent roughness and very permeable soils. The RSM-GL model simulates an extensive canal network. This network includes all primary canals that are maintained by the SFWMD. It also includes several secondary canals that are of importance. In addition, the model uses the Water Control District (WCD) feature available in the RSM to simulate some secondary and tertiary canals as well. Relevant structure operations associated with the WCDs and the canal network are simulated using the functionality available in the model. Only the surficial aquifer is simulated in the RSM-GL model. The RSM-GL was not developed to simulate deep groundwater flows. Other surface water models, including the SFWMM, have used similar approaches. The WCAs and ENP contain a significant peat layer that affects stages within those areas. This surficial peat layer is simulated explicitly in the RSM-GL using a stage-volume converter feature that is unique to the RSM. The model-domain contains several hundred Public Water Supply (PWS) wells that tap the surficial aquifer. The model-domain contains several roads and levees that act as overland flow barriers. The canal and regional groundwater seepage contributions across these levees are explicitly simulated in the RSM-GL model.

The RSM-GL application of the RSM was specifically calibrated to support the evaluation of proposed project features for the CERP WCA-3 Decompartmentalization and Sheetflow Enhancement project (Decomp). The RSM-GL model has been previously applied by the SFWMD/USACE to support base condition modeling and evaluations of the final array of alternatives for the Decomp Project Implementation Report 1 (PIR 1) during 2010-2011. Both the Decomp and CEPP projects are components of the CERP, and the features of the prior Decomp project are central components to the

CEPP. The Decomp modeling strategy selection of RSM-GL as a preferred sub-regional hydrologic modeling tool was significantly leveraged during identification of the CEPP modeling strategy.

An extensive modeling strategy development and review effort was used by the Decomp project delivery team (PDT) to identify the RSM-GL model as the preferred sub-regional modeling tool to support Decomp PIR 1 alternative evaluations. The comprehensive Decomp modeling strategy was endorsed by the CERP Interagency Modeling Center (IMC) in 2008; the IMC, under its responsibility to serve as a central point to coordinate CERP and CERP-related modeling activities, is routinely consulted to implement peer reviews of models and their applications. In addition, IMC peer review of the available sub-regional hydrologic modeling tools resulted in an IMC recommendation for Decomp application of the RSM-GL model. As with other peer review requests for CERP applications, the peer review scope requested the peer review panel to judge the adequacy of the RSM-GL model with respect to model data needs, model spatial and temporal resolutions, model documentation, model capabilities, model limitations, and the theory upon which it is based. The goals of this IMC peer review request were two-fold: (1) to ensure that the RSM-GL model was developed and implemented based on sound science and modeling principles; and (2) to determine the suitability of the RSM-GL to support Decomp PIR 1 plan formulation and evaluation.

The IMC peer review report recognized that the RSM-GL as: (1) an improvement over the SFWMM with respect to model methodology (surface flow, canal flow, sub-surface flow, evaporation, evapotranspiration, infiltration and seepage), especially when considered as a sub regional tool for implementation of the hydrologic portion of the DECOMP modeling strategy; (2) an improvement over the SFWMM with respect to the model grid density and local grid refinement capabilities, and modeler control of adding, removing, or modifying canals, structures, and structure operation rules; (3) able to distinguish between spatial and temporal differences in water depths/stage (including recession rates), and overland and canal flow adequately for CERP evaluations; and (4) if used with caution and adequate interpretation of the model output, able to show some important differences between the varying degrees of Miami Canal backfill or plugging that may be proposed for Decomp PIR 1 alternatives.

The IMC peer review report summarized the overall strengths of the RSM-GL, focusing on Decomp PIR 1 model needs:

- The model is capable of predicting the intricate results of implementing physical and operational alternatives. It can be used to simulate the complexity of integrated surface water and groundwater systems under natural conditions and to support decision-making and management operations.
- The model is capable of simulating rainfall, evapotranspiration, irrigation, crop water demand, and groundwater withdrawals in the surficial aquifer system. These processes are critical to South Florida.
- The model has physically-based formulations for the simulation of overland/ sheet flow, groundwater flow, evapotranspiration, infiltration, levee seepage, irrigation, urban water use, storm water detention, river/canal flows, and structure flows.
- The model is capable of simulating the unique features of low-relief topography in south Florida, including the interactions between surface water and groundwater, levee-canal systems, and complex structure operations such as well pumping rate, gate opening and closing, and flow diversion between management water control units according to established management rules.

- The model uses an irregular triangular mesh so the model boundaries and features can be accurately defined. The triangular mesh system can be designed to conform to all important features, boundaries, roads and levees.

The IMC peer review report also summarized the overall weaknesses of the RSM-GL, focusing on Decomp PIR 1 model needs:

- The results of model calibration and verification indicate large errors in flow computation at structures. The model also performs poorly in predicting canal stages at several of these locations.
- Water quality is also an important issue in CERP projects. At this time, the RSM model is not capable of simulating water quality.
- In the current setup, most of the structure flows are imposed as internal boundary conditions. Therefore, the model cannot be considered calibrated to surface water flows.
- The current mesh discretization, while fine for regional-scale modeling, may not be adequately detailed to evaluate the features of Decomp PIR 1. It should be noted that the RSM Glades LECSA Model was not developed solely for the DECOMP project application, but rather for broader CERP applications. Therefore the need for such refinements is to be expected.
- The model does not recognize separate aquifer layers and emulates groundwater flow processes as one hydrologic unit, which may be acceptable only for coarse spatial discretization and flow regimes with hydrostatic pressure distributions. This may or may not be a significant weakness depending on the nature of the Decomp PIR 1 alternative scenarios to be evaluated. It should also be mentioned that the model assumes a no-flow boundary condition at the base of either the Biscayne / Gray Limestone aquifer in the Lower East Coast area or the surficial aquifer in Lower West Coast area. This assumption is generally acceptable, but may yield significant error in localized areas of significant hydrologic stresses (e.g., well fields).
- Although the model is generally well-calibrated to water levels, the model seems to have problems predicting extreme low and high water levels, especially the low stages. Considering these low stages over extended time periods could be critical to aquatic life and the ecological system, this weakness is of importance and should be improved if possible. Or at a minimum, this weakness should be considered when interpreting model results.

Key priority recommendations from the IMC peer review panel for enhancements to the RSM-GL model and the calibration/validation report content, as agreed upon following a May 2008 meeting with SFWMD, USACE, IMC managers, and the IMC peer review panel, were implemented for the final version of the RSM-GL that was applied for Decomp PIR 1 regional modeling and, ultimately, CEPP. As a result of the RSM peer review recommendations, a finer resolution mesh was developed for the Decomp PIR 1 project area, and additional sensitivity testing and model output statistics were completed for incorporation into the RSM-GL calibration/verification report. The peer review recommendations and IMC endorsement of the RSM-GL model also initiated a significant two year coordination effort between the Decomp Ecological and Water Quality sub-team and the SFWMD RSM model developers to develop a robust Decomp PIR 1 evaluation methodology, including efforts to port existing SFWMM performance measure tools to RSM, development of new indicator regions and aggregation methods, and extensive performance measure testing and validation.

The final Decomp RSM-GL calibration/verification report includes documentation of model enhancements which were recommended by the IMC model peer review panel and subsequently agreed to by the SFWMD RSM development team. The complete RSM-GL model calibration and verification report, which was completed in December 2011 as part of the Decomp PIR 1 project modeling effort, is posted with the Decomp project documentation report (Annex A-1, Appendix B-11): http://www.evergladesplan.org/pm/projects/docs_12_decomp_doc_report.aspx. The reader should refer to this documentation for a more complete review of the RSM-GL model development, calibration methods, and calibration/verification performance and statistics. The CEPP utilized the RSM-GL model version that was developed and thoroughly documented for the Decomp PIR 1 project, while additionally utilizing an extended 1965-2005 period of simulation (Decomp modeling used 1965-2000; no additional verification was completed for the 2001-2005 period for CEPP application of the RSM-GL). Some minor localized improvements to the Decomp RSM-GL model were also included to improve the capability of the RSM-GL to more effectively represent critical CEPP project components, with these enhancements extensively tested and confirmed to not significantly alter the previous calibration and verification performance; details of these improvements and testing results are documented in the CEPP Model Documentation Report (MDR) for the CEPP Model Supported Screening Efforts (MDR 1), which is included in the CEPP Final PIR as Annex A-3 of this Appendix. CEPP MDRs were jointly prepared developed by the SFWMD Hydrologic and Environmental Systems Modeling (HESM) and the IMC.

Following the CEPP announcement in October 2011, the USACE SAJ and the SFWMD decided to integrate the previous Decomp planning effort into the CEPP. SAJ prepared a documentation Report to summarize the Decomp plan formulation and evaluation efforts, information obtained by the planning team, engineering work efforts, and lessons learned to date. The Decomp documentation report was used by the CEPP team and is available to staff and managers involved in the interagency state-federal Everglades restoration program as a resource to guide future planning efforts. The report documents the plan formulation and evaluation of seven alternatives (subset of final array), all plan formulation activities leading up to the development of the final array of alternatives, recommendations for an adaptive management strategy, and application of extensive hydrologic modeling (including RSM-GL application) conducted to support the formulation and evaluation efforts. The Decomp documentation report can be reviewed at the following location:

http://www.evergladesplan.org/pm/projects/docs_12_decomp_doc_report.aspx

The Hydrology and Hydraulics Annex to the Engineering Appendix (Annex A-1) of the Decomp documentation report provides comprehensive documentation of the technical support provided by the SAJ Water Resources Engineering Branch: hydrologic data collection and analyses; development and application of numerical modeling tools to support PDT evaluations; preliminary hydraulic design efforts; and additional work-in-progress technical information for consideration by future CERP planning efforts.

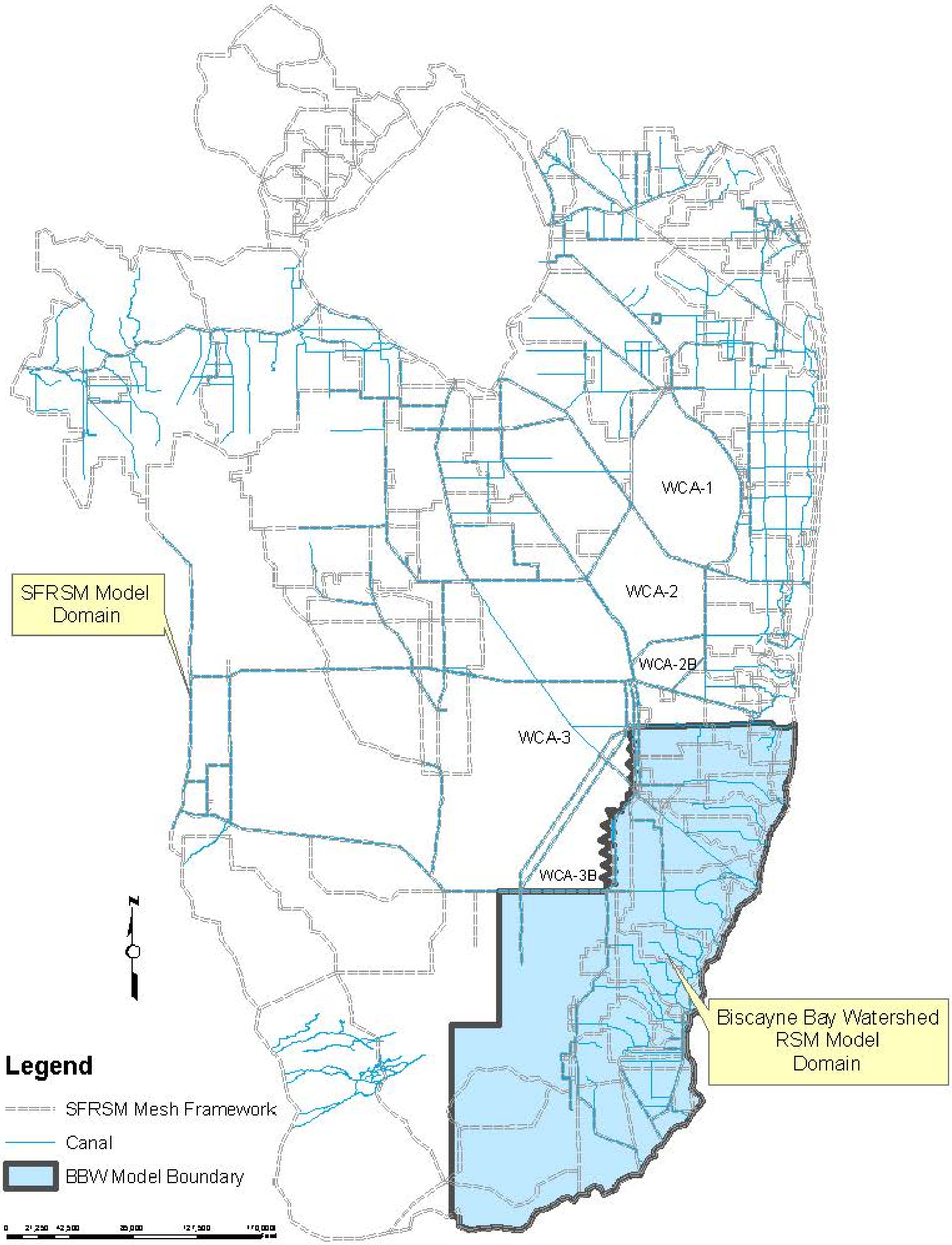


FIGURE A.8- 1: BBCW/C111 RSM MODEL DOMAIN

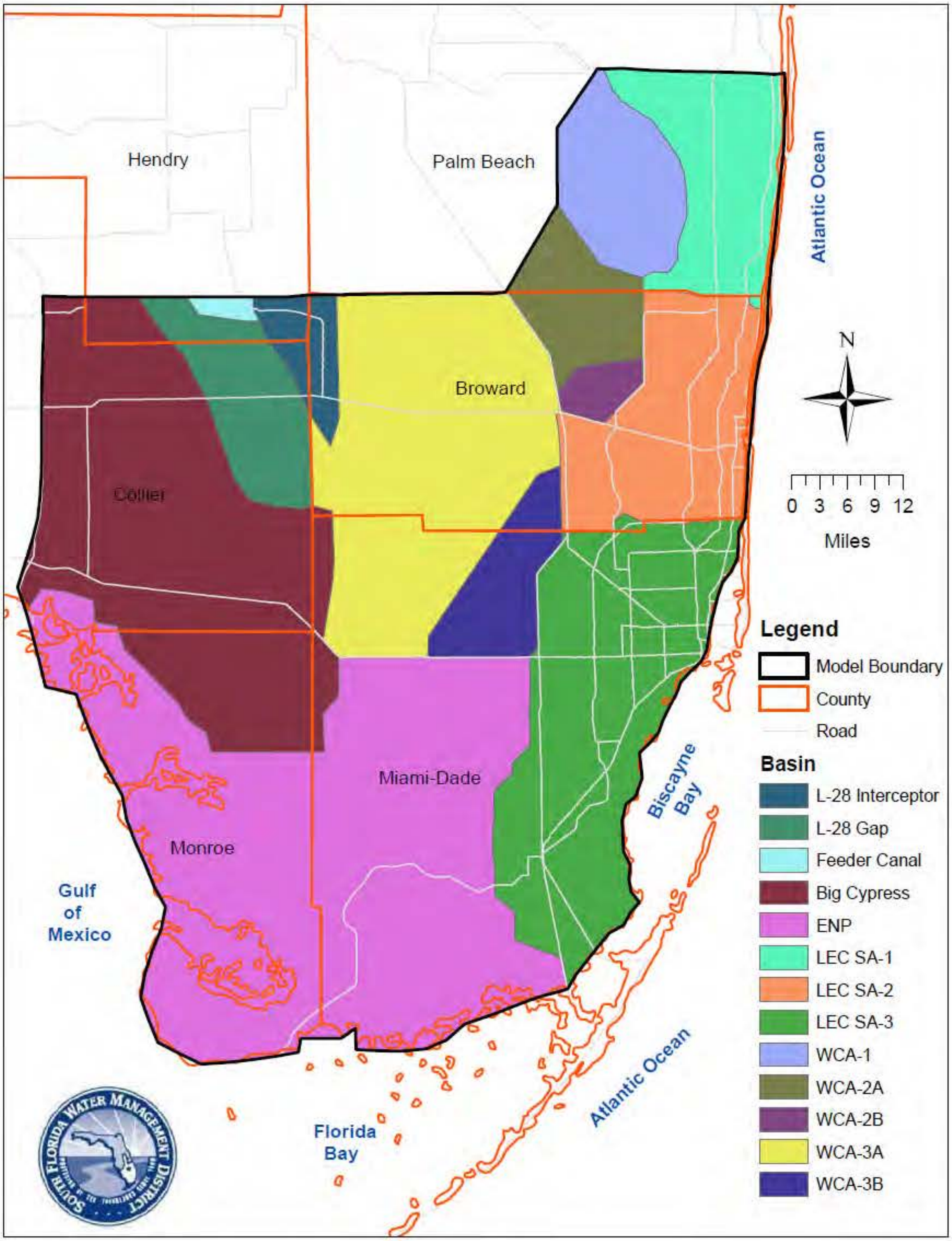


FIGURE A.8-2: RSM-GL MODEL DOMAIN

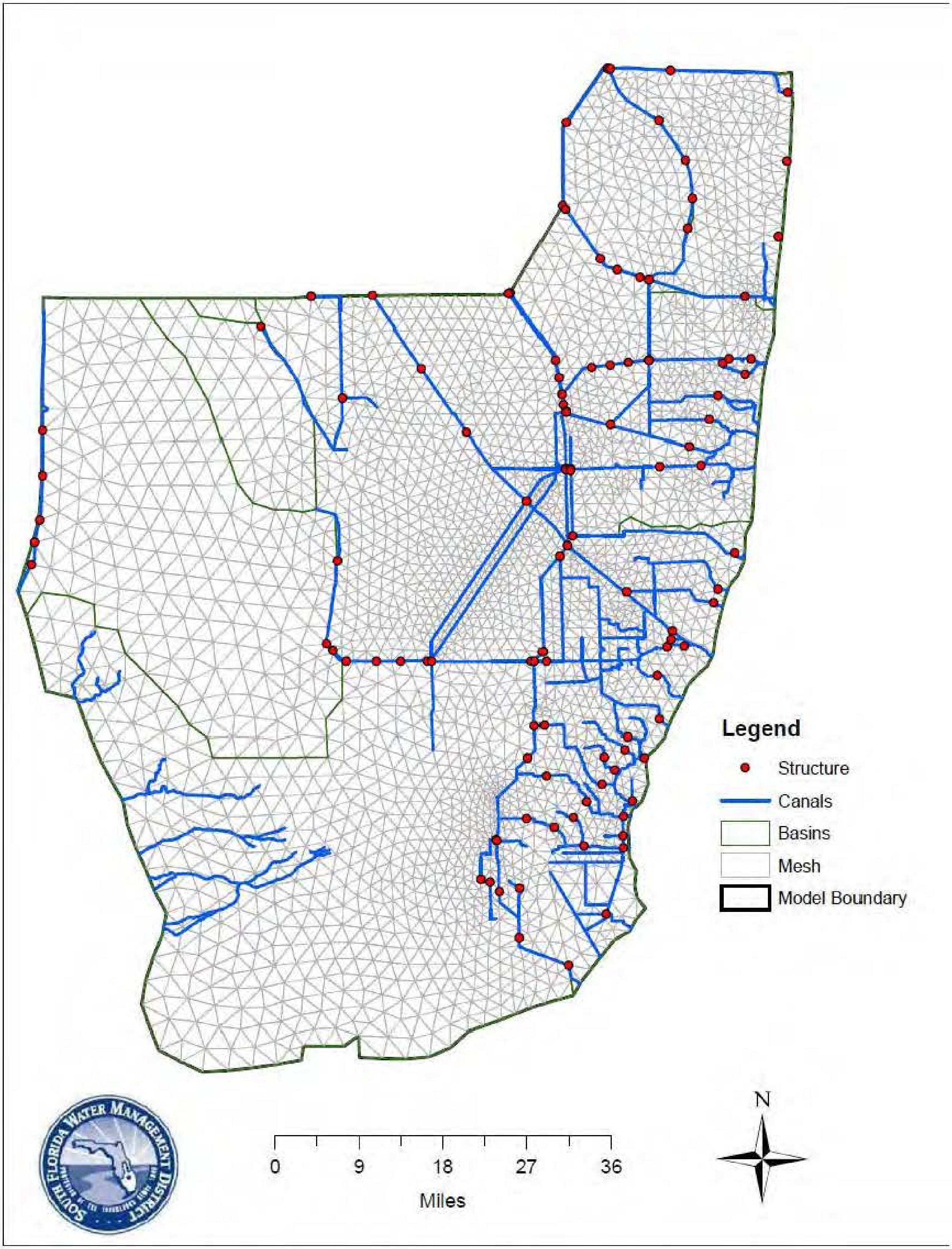


FIGURE A.8-3: RSM-GL MODEL MESH, CANAL NETWORK, AND SIMULATED WATER CONTROL STRUCTURES

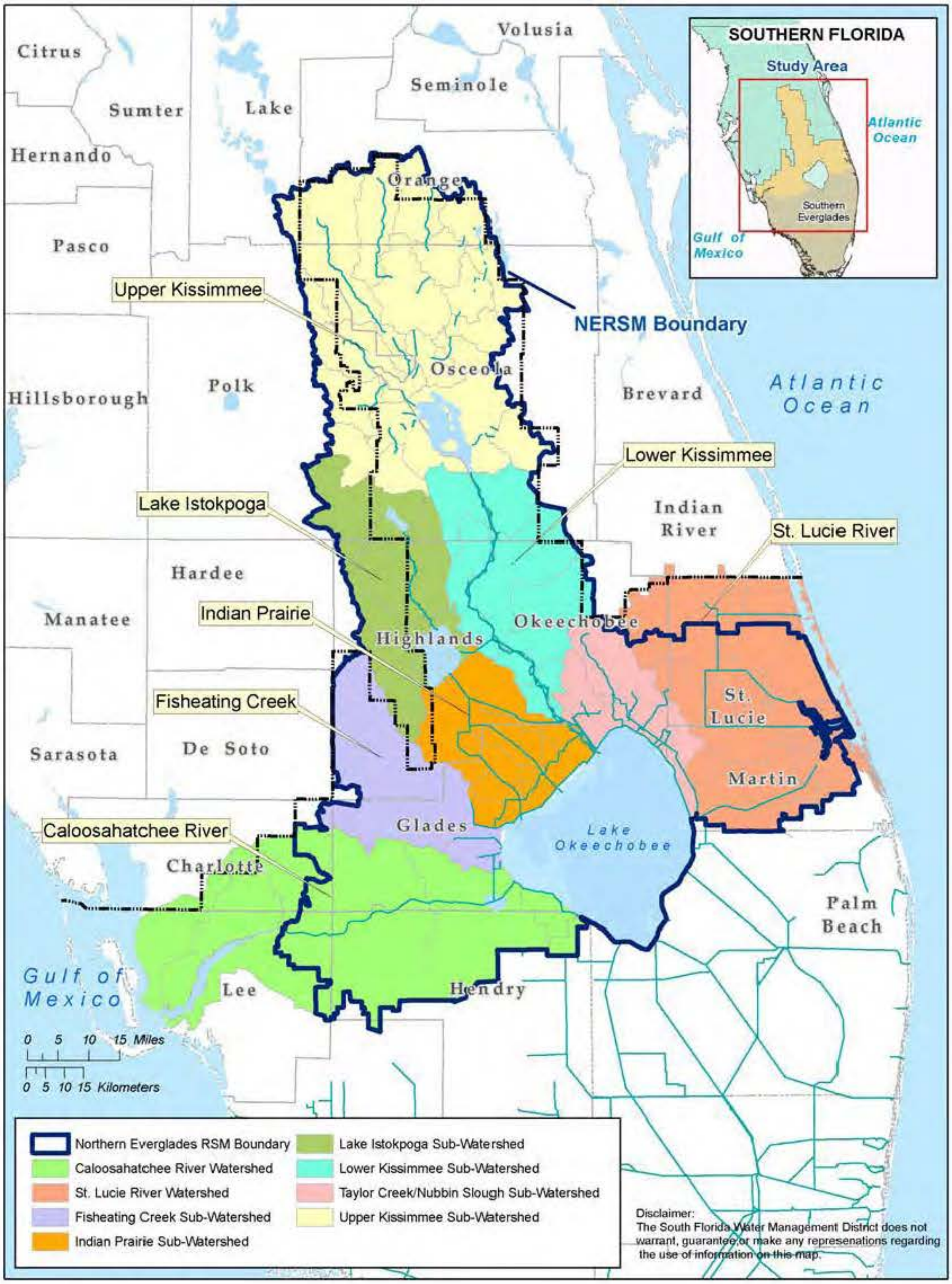


FIGURE A.8-4: NERSM MODEL DOMAIN

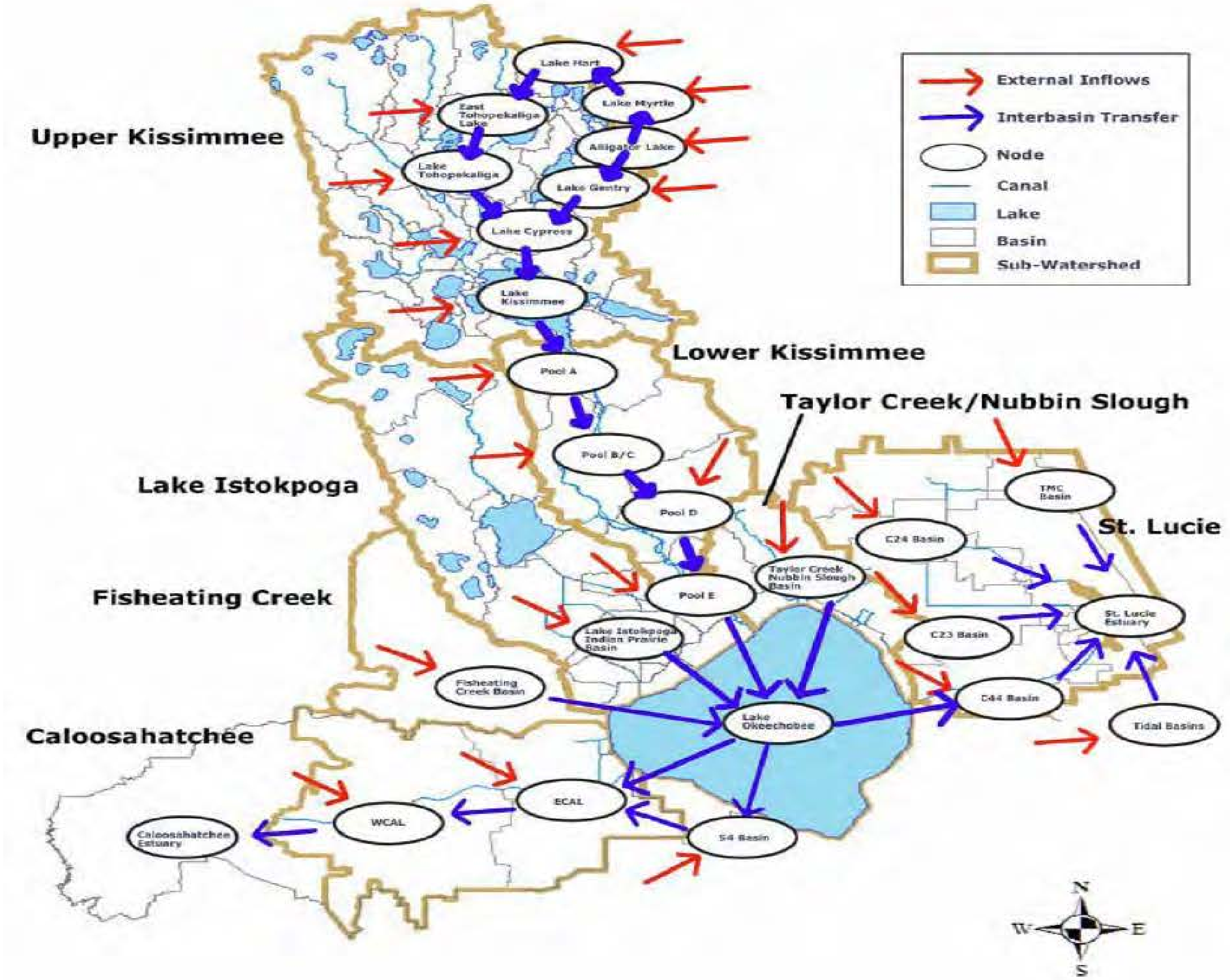


FIGURE A.8-5: NODE-LINK DIAGRAM REPRESENTATION OF NERSM (RSM-BN NORTHERN DOMAIN)

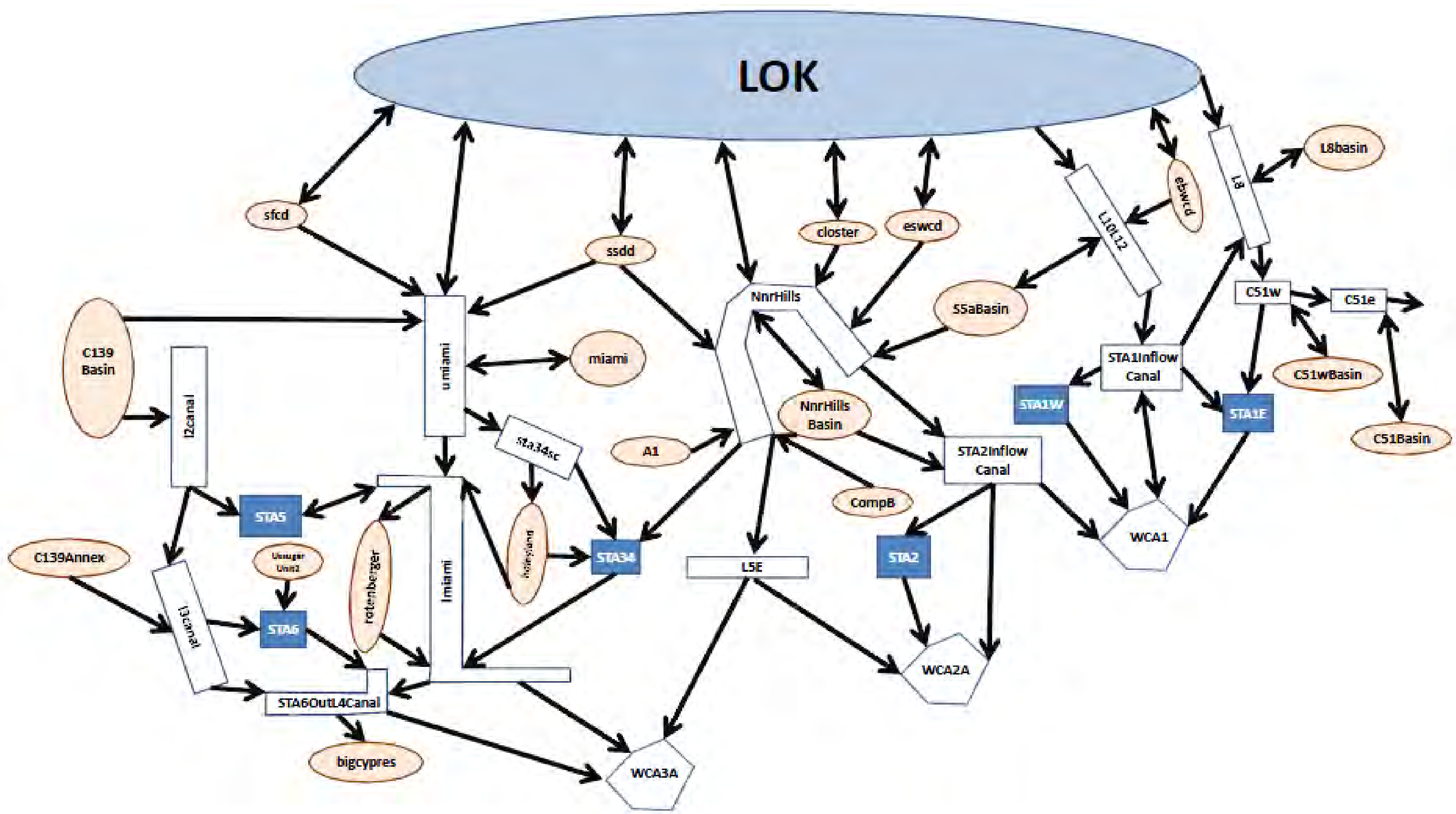


FIGURE A.8-6: NODE-LINK DIAGRAM REPRESENTATION OF EAA FOR RSM-BN EXISTING CONDITION BASELINE

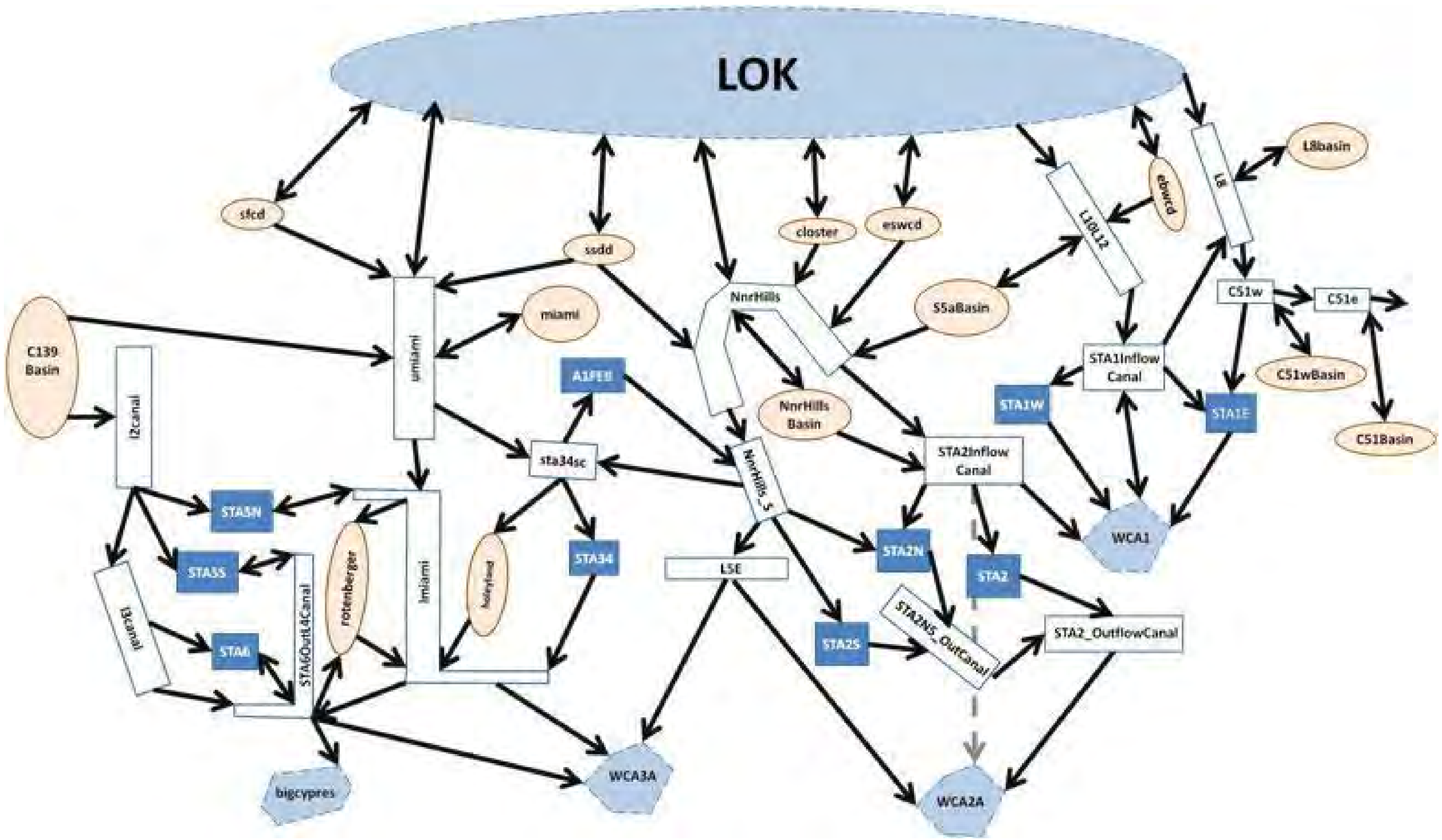


FIGURE A.8-7: NODE-LINK DIAGRAM REPRESENTATION OF EAA FOR RSM-BN FUTURE WITHOUT PROJECT CONDITION BASELINE

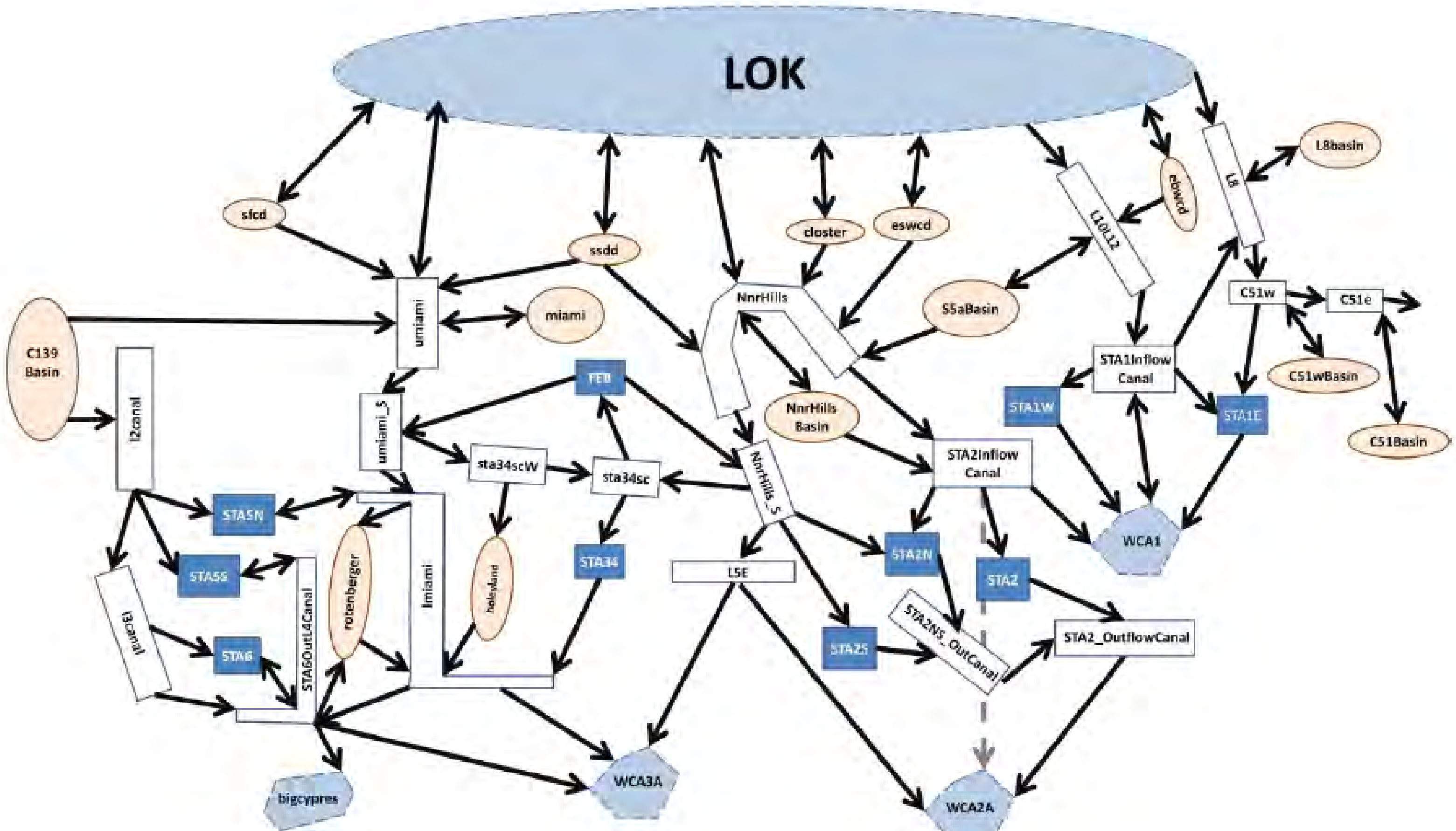


FIGURE A.8-8: NODE-LINK DIAGRAM REPRESENTATION OF EAA FOR RSM-BN ALTERNATIVES

A.8.1.3 Modeling Tool Overview: Hydrologic Engineering Centers' River Analysis System (HEC-RAS)

HEC-RAS modeling tools are utilized to evaluate potential canal conveyance modifications and structural modifications identified with the CEPP Recommended Plan components. HEC-RAS is also applied as a hydraulic design tool to aid with design of new gravity water control structures. Detailed documentation of the CEPP hydraulic design methods and results are provided in Sections A.5.3, A.7.3, and A.7.3 of the Engineering Appendix.

HEC-RAS is an integrated package of hydraulic analysis programs, in which the user interacts with the system through the use of a Graphical User Interface (GUI). The system is capable of performing steady and unsteady flow water surface profile calculations, sediment transport/movable boundary computations, water quality analysis, and several hydraulic design computations. Hydraulic losses through the channel, bridge, culverts, spillways and other hydraulic structures can be modeled in both the steady state and unsteady state modules. The unsteady flow component is capable of simulating one-dimensional unsteady flow through a full network of open channels. Special features of the unsteady flow component include dam break analysis, levee breaching and overtopping, pumping stations, navigation dam operations and pressurized pipe systems. These model features can be useful in identifying conveyance deficiencies in canals and structures under steady state or dynamic flow conditions for particular flood events.

The HEC-RAS includes capabilities that allow the model to apply complex operation of gated structures and pump stations. HEC-RAS is capable of simulating interaction between 1-dimensional channel flow and 2-dimensional floodplain flow, allowing for more accurate floodplain mapping. In areas where the interaction of open channel flow and aquifer groundwater needs to be explicitly modeled, a new integrated tool based on the original HEC-RAS and MODFLOW models can also be used to accurately simulate the aquifer/canal flow exchange.

For an Unsteady Flow analysis, there are several different boundary conditions available including flow hydrographs, stage hydrographs, stage and flow hydrographs, and rating curves. Boundary conditions must be imposed at all external model cross sections and can be added to any desired internal location.

Boundary conditions for stage and flow are available from either historical data or from the RSM-BN or RSM-GL sub-regional hydrologic models; however, HEC-RAS simulations developed for CEPP will be steady state and not reliant on time series output from the hydrologic modeling efforts.

For CEPP, the HEC-RAS computer software model was utilized for all 1-dimensional hydraulic routings at the localized scale, where necessary to develop hydraulic design criteria for cost estimating design purposes and further engineering design in PED. Cost estimating design purposes examples include the following: (1) identifying type and size of water control structure, (2) geometry of canals and other conveyances, (3) pump head requirements, (4) levee crest elevations, and (5) miscellaneous requirements for civil, geotechnical, and structure designs. The strength of HEC-RAS is the continual updates made as the science of computational hydraulic engineering progresses, which adds additional accuracy, precision, and stability, thereby confidence with consistent peer reviewed model results, dependent on input. HEC-RAS is currently used worldwide for these reasons. The one real weakness of HEC-RAS is that it is constrained to 1-dimensional flows versus 2-dimensional or 3-dimensional flows. During PED, where determined necessary, a 2-dimensional computer software model (likely ERDC's Adaptive Hydraulic model, AdH-2D) will be used to model conveyance paths to determine final hydraulic design details; such areas include structure discharge basins, the FEB interior levee (baffle), and others to be identified later.

A.8.2 Preliminary Screening

A.8.2.1 Summary of Screening Tools and PIR Documentation

Execution of the CEPP modeling strategy and application of the initial recommended modeling tool suite initiated in January 2012. Preliminary screening assessments for Lake Okeechobee, the northern estuaries, and the impoundment storage within the EAA, collectively referred to as the "North of the Red Line components," utilized the RESOPS model, the LOOPS model, and the C-43 Spreadsheet Model. The CEPP plan formulation approach, screening methods, and results for the North of Red Line components, which ultimately identified the ~14,000 acre Flow Equalization Basin (FEB) on the EAA A-2 site for inclusion in the CEPP Recommended Plan, are summarized in **Section 3** of the CEPP PIR main report. Formulation and screening analysis for the EAA storage component of CEPP was completed between January and July of 2012. Preliminary screening assessments for the WCAs and ENP, collectively referred to as the "South of the Red Line components," utilized the iModel optimization tool and limited-scope sensitivity simulations using the RSM-GL. The CEPP plan formulation approach, screening methods, and results for the South of Red Line components, which ultimately identified the remaining CEPP Recommended Plan components for the L-4/L-5 Levees, Miami Canal, L-67A/L-67C Levees, L-29 Levee, L-67 Extension Levee, and L-31N Canal within WCA-3 and ENP, are also summarized in **Section 3** of the CEPP PIR main report. Formulation and screening for the Red, Green, Blue, and Yellow Line CEPP components were primarily completed between June and November of 2012. Further documentation of the CEPP screening results and formulation approach is not included in the Engineering Appendix. The CEPP modeling strategy provides an overview of the modeling tools, including maps of the model domains, applied throughout the plan formulation process and how the tools were applied in support of the CEPP planning process (refer to Reference 1, included in the Hydrologic Modeling Annex A-2).

For the final array of alternatives, analysis of the North of Red Line components and the South of the Red Line components were conducted using the RSM-BN and the RSM-GL, respectively. This Engineering Appendix and the supporting Hydrologic Modeling Annex A-2 provide documentation of USACE SAJ performance analysis of the hydrologic modeling results for the CEPP final array of alternatives only, with specific emphasis on engineering design considerations that were actively tracked throughout the CEPP formulation, preliminary screening, and alternative development efforts. HESM/IMC Model MDR 1 (Annex A-3) reviews the various model-supported feature screening efforts undertaken at various points in the planning process.

A.8.2.2 Decomp RMA-2 Screening of Miami Canal Plug Configurations

General overview information and summary conclusions from the Decomp RMA-2 screening analysis, which were utilized by the CEPP plan formulation efforts, are documented in this section, with further detailed information provided in the Hydrologic Modeling Annex A-2.

The Decomp project conducted a screening model evaluation of numerous Miami Canal plug configurations (plug length and spacing) to identify the optimal configuration(s) which most closely mimic the performance of a complete/full Miami Canal backfill within WCA 3A. The analysis considered both the use of existing fill onsite and importing additional fill to the project from offsite. Due to the limitations of the RMA-2 screening tool, plug configurations were also evaluated with the higher resolution model RSM-GL as part of the final array of alternatives.

The RMA-2 and RSM-GL modeling efforts conducted for Decomp indicated that plugs along the Miami Canal may have the potential to work as effectively hydrologically as full backfill to reduce drainage and the disruption of sheetflow caused by the Miami Canal. RSM-GL final array modeling during Decomp also revealed that potential benefits from backfilling the Miami Canal south of I-75 were limited under Decomp PIR 1 assumptions (particularly redistribution of existing inflows to WCA-3A only and limited MWD outlet modifications for WCA-3A), probably due to the limited conveyance out of WCA 3A resulting in continued ponded conditions in southern WCA 3A. The 2012 Decomp PIR 1 project documentation report recommended that proposed alterations to the Miami Canal south of I-75 should be reevaluated if the ponding conditions within southern WCA 3A were altered or alleviated.

Although the CEPP Recommended Plan proposes significant increased conveyance between WCA-3A, WCA-3B, and ENP as compared to the Decomp formulation assumptions, and although the CEPP final array modeling indicates significant reduction to the frequency and magnitude of ponded conditions within southern WCA-3A, no meaningful plan formulation effort was given to modifications to the Miami Canal south of I-75 because the CEPP plan formulation for the WCA-3A hydropattern restoration and Miami Canal components significantly leveraged the previous Decomp formulation efforts. Given consideration of CEPP schedule limitations and based on the results of the CEPP preliminary screening efforts (refer to **Section 3** of the CEPP PIR main report for detailed discussion of the formulation methodology), CEPP preliminary screening modeling conducted with the RSM-GL in July 2012 evaluated only one option for Miami Canal modifications south of I-75 – inclusion of a 4000 foot long plug centered at S-340 and an 8000 foot long plug starting south of the C-11 Extension. The CEPP RSM-GL screening modeling additionally was conducted as a sensitivity analysis starting with the final array modeling from Decomp with WCA-3A inflows increased to account for the approximately 20 percent increase assumed for CEPP. Therefore, since the CEPP screening modeling assumptions incorporated the MWD project outlet modifications for WCA-3A, the screening modeling results did not demonstrate the expected significant reduction to the frequency and magnitude of ponded conditions within southern WCA-3A that would be realized if the CEPP components identified along the Green Line and Blue Line had been included for the CEPP screening. A different set of CEPP screening assumptions may have demonstrated increased benefits associated with the Miami Canal modifications south of I-75, but these analyses will instead be shifted for future consideration in future CEPP increments. .

The plug proposed in the southern reach of the Miami Canal was intended to reduce the drainage effect of the Miami Canal, south of the existing S-340 structure. The Miami Canal south of S-340 and the L-67A Canal currently provides approximately 30 miles of unobstructed southerly canal flow towards the WCA-3A outlet structures along Tamiami Trail (S-333 and the S-12s), and the Miami Canal is aligned parallel to the northwest-to-southeast direction of flow within WCA 3A. In addition, initial screening modeling conducted during Decomp indicated that hydrologic performance improvements within Northeast WCA-3A were generally best achieved through backfill of the South Miami Canal Segment. Effects to recreational access were considered during CEPP formulation of the Miami Canal southern plugs, and the proposed plug location was south of the junction of the Miami Canal/C-11 Extension and north of the Holiday Trail from Everglades Holiday Park. Recreational access from Everglades Holiday Park to the Miami Canal between S-340 and the proposed plug, to the Miami Canal south of the proposed plug, and to the L-67A Canal would be maintained with this proposed configuration. Based on review of aerial photographs, the plug length was proposed at 8000 feet, starting south of the C-11 Extension. The source of backfill material for the proposed plug was envisioned as the nearby spoil mounds along the

Miami Canal and the then CEPP-proposed spoil mound degrade/gaps along the C-11 Extension (this component was subsequently excluded with the CEPP final array), with the proximity of the C-11 Extension spoil material serving as a factor in the plug location selection. Based on preliminary surveys of the Miami Canal spoil mound material under Decomp, a maximum of approximately 5.5 of the 9.7 miles (57%) of the Miami Canal between S-340 and the L-67A Canal could be backfilled with the on-site spoil material.

The Miami Canal plug configuration was ultimately screened out from the CEPP final array components because the RSM-GL screening modeling demonstrated only localized dry year benefits for the single evaluated plug configuration, which could not justify the additional incremental cost of approximately $13 million. However, consistent with the Decomp report conclusions, the final conclusions identified from the CEPP screening assessment should include consideration of the assumptions related to limited relief for the ponding conditions in southern WCA-3A and the limited spatial extent of plugs which were evaluated. Given recognition of this context, consideration of Miami Canal modifications south of I-75 will likely warrant further detailed evaluation for future CERP/CEPP increments.

Beyond the insights afforded by hydrologic modeling, as further summarized in the Decomp report, questions remain regarding the ability of plugged canals to function ecologically as the pre-drainage ridge and slough landscape, especially in low flow conditions, and what the continuing effect of deep holes (spaces between plugs) in the canal have on Everglades flora and fauna, including providing pathways for invasive exotic species. These uncertainties would need further assessment for consideration of future plug options for canals within the Greater Everglades, although additional information may also be realized through CEPP adaptive management strategies.

Although the Miami Canal plugs were not included in the components for the CEPP final array (all final array alternatives included complete backfill of the Miami Canal to I-75, starting from either approximately 1.5-2.0 miles south of S-8 (Alternative 1, Alternative 4R, and Alternative 4R2) or immediately downstream of S-8 (Alternatives 2 through 4)), information from the Decomp RMA-2 plug analysis was additionally utilized to establish the initial proposed spacing between Miami Canal mounds because the Miami Canal backfill to bedrock grade will leave remnant open water segments between the mounds that are expected to behave hydrologically similar to the plug options that were evaluated with RMA-2 for Decomp. As further documented in Hydrologic Modeling Annex A-2, overall plug performance (compared to the full backfill condition) is significantly diminished for plug spacing scenarios greater than approximately 4000-6000 feet, whereas no observed similar trend is observed for plug length. The initial proposed spacing between Miami Canal mounds was selected at 1 mile (5280 feet), given consideration of the insights from the Decomp RMA-2 modeling and overall CEPP project cost considerations (increased cost with reduced distance between mound features).

The Decomp modeling strategy proposal recommended a limited modeling effort, utilizing a fine resolution hydraulic modeling tool, to allow evaluation of the potential near-field effects for Miami Canal backfill options and yield enhanced understanding about the effectiveness and impacts of each type of canal backfilling option. The need to simulate three dimensional flow fields was not a critical element of the Miami Canal local feature modeling effort; two-dimensional flow fields with depth-averaged velocity parameters (including within open canal segments) were determined to provide sufficient analysis for the stated scope of this effort, noting the shallow depths representative of typical overland flow in the project area. RMA2 was recommended by the USACE within the Decomp modeling strategy as the most appropriate tool for this analysis. RMA2, developed by the Resource Management

Associates (RMA), has been previously reviewed and classified by the HH&C CoP as a "CoP Preferred" hydraulic design and river hydraulics modeling tool.

A.8.3 Evaluation of the Final Array of Alternatives

A.8.3.1 Baseline Condition Modeling

The study area for the CEPP encompasses Lake Okeechobee, the Northern Estuaries (St. Lucie River and Indian River Lagoon and the Caloosahatchee River and Estuary), a portion of the EAA, the WCAs, ENP, the Southern Estuaries (Florida Bay and Biscayne Bay), and the Lower East Coast (LEC). **Section 2.4** of the CEPP PIR main report provides a summary description of the existing and future without project conditions within the study area. Detailed documentation of existing and future without project conditions is further provided in Appendix C.1 to the CEPP PIR main report, including detailed documentation of hydrology, regional water management, flood control, and water supply performance for each base condition. Selected graphics are included to illustrate the performance of each baseline condition.

Hydrologic modeling simulations of the existing condition baseline (ECB) and the CEPP future without project condition (FWO) were developed with the RSM-BN and RSM-GL sub-regional modeling tools, to provide baseline conditions for plan formulation and the assessment of CEPP project benefits and the preliminary assessment of CEPP alternative performance for the level-of-service for flood protection and water supply (ECB). The ECB was developed to represent the system-wide infrastructure and operations that were in place at the time CEPP plan formulation was initiated, approximately January 2012. The FWO for CEPP assumes the construction and implementation of currently authorized C&SF and non-CERP projects, and other Federal, state or local projects constructed or approved under existing governmental authorities that occur in the CEPP study area; the CEPP FWO therefore included first generation CERP projects already authorized and under construction (Indian River Lagoon-South Project, Picayune Strand Restoration Project, Site 1 Impoundment Project), second generation CERP projects still pending Congressional authorization (Biscayne Bay Coastal Wetlands Project, Broward County Water Preserve Areas Project, Caloosahatchee River (C-43) West Basin Storage Reservoir, C-111 Spreader Canal Western Project), and non-CERP projects currently in progress (SFWMD Restoration Strategies, C&SF C-51 West End Flood Control Project, the C-111 South Dade Project, the Kissimmee River Restoration Project, Modified Water Deliveries, and the Department of Interior (DOI) Tamiami Trail Modifications Next Steps Project. Documentation of RSM-BN and RSM-GL assumptions for the ECB and FWO baseline conditions are provided in Reference 2 of the Hydrologic Modeling Annex A-2, respectively.

The CEPP PIR report documentation and two complete sets of RSM-BN and RSM-GL hydrologic model performance measure output are posted on the Everglades Plan public web site for the CERP:
http://www.evergladesplan.org/pm/projects/proj_51_cepp.aspx

The following complete performance measure data sets are provided to facilitate additional review of the hydrologic modeling output for the baselines and the Recommended Plan, Alternative 4R2:

- ECB, FWO, Alternative 4R, Alternative 4R2 (comparison used for NEPA evaluation in Section 5 of the main PIR report)
- ECB, 2012EC, IORBL1,Alternative 4R2 (comparison used for the Savings Clause and Project Assurances evaluation in Annex B of the PIR report)

For additional discussion of the final array modeling and the baseline updates to support the Savings Clause evaluation, refer to **Section A.8.3.2** and **Section A.8.3.2.5**, respectively. HESM/IMC MDR 2 (Annex A-3) reviews the model representations of existing conditions and the future without project conditions used throughout the CEPP plan formulation efforts.

Final CEPP hydrologic modeling products have been uploaded to the CERP Model Management System (MMS), a geographic information system (GIS) based application that includes model input data, select model output data, source code/executable files and documentation. CEPP modeling products in MMS can be accessed directly at the MMS project page through the Everglades Plan public web site:
http://cerpmap1.cerpzone.org/arcgisapps/CERPMMS/CerpReport/ProjectReport.aspx?projectID=687

A.8.3.2 Final Array Modeling

CEPP plan formulation efforts identified the final array of four alternatives (Alternatives 1 through 4) in November 2012, and the corresponding RSM-BN and RSM-GL simulations of the alternatives was subsequently completed in December 2012. HESM/IMC MDR 3 (Annex A-3) reviews these four proposed with-CEPP project model representations examined during this first round of CEPP plan formulation. As documented in **Section 4.6** of the CEPP PIR main report, modifications to the final array were identified during January-February 2013 that resulted in the identification of Alternative 4M as the National Ecosystem Restoration (NER) Plan. The evaluation also identified the need to revise the operations for Alternative 4M, which was not evaluated with hydrologic modeling, to ensure the project savings clause constraints are met, to minimize localized adverse ecological effects, and to identify additional opportunities to provide for other water related needs.

Three additional modeling scenarios were conducted in the following months to identify project effects resulting from the identified operational changes: Alternative 4R (completed February 2013), Alternative 4R1 (May-June 2013), and Alternative 4R2 (June 2013). The first refinement, Alternative 4R, focused on operational changes to avoid potential impacts to water supply levels of service in the Lake Okeechobee Service Area (LOSA) and the LEC. Refinements included alleviating potential ecological impacts from lowered water depths in WCA 2B by retaining a small portion of the water in WCA 2B that Alternative 4M had diverted to WCA 3A. Increases in low flow events to the St. Lucie Estuary, minimized reductions in freshwater flows to Biscayne Bay, and improved water depths in eastern WCA 3B for purposes of improving environmental conditions were also considered. The Alt 4R refinement resulted in an alternative that lessened concerns over violating constraints yet there remained room for improvement in LOSA water supply and the spatial distribution of groundwater and canal discharges in the LEC to provide greater confidence in meeting legal requirements of the savings clause. Building on the performance improvements achieved with the Alternative 4R operational changes, Alternatives 4R1 and 4R2 increased public water supply demand for Lower East Coast Service Area 2 (LECSA 2 - Broward County) and Lower East Coast Service Area 3 (LECSA 3 - Miami-Dade County) to determine whether there was a threshold for increased public water supply demand that would be capable of balancing increased water supply demands for LECSA 2 and LECSA 3 with maintaining the natural system performance of Alternative 4R. Alternative 4R1, which increased public water supply demand by 19 million gallons per day (MGD) for LECSA 2 and 53 MGD for LECSA 3, was not assessed in detail in the PIR report due to significant performance concerns identified with the observed reductions in discharges to Biscayne Bay and increased risk of saltwater intrusion at several wellfield locations. Based on information gained during the modeling of Alternative 4R1 and related RSM-GL sensitivity simulations, the subsequent Alternative 4R2 simulation limited the increase to public water supply demand by 12

MGD for LECSA 2 and 5 MGD for LECSA 3 and was determined to be successful with maintaining the ecological performance of Alternative 4R without the negative effects to LEC groundwater and Biscayne Bay that Alt 4R1 realized. Alternative 4R2 was identified in the PIR main report as the Recommended Plan. HESM/IMC MDR 4 (Annex A-3) reviews the model representation of the CEPP Recommended Plan and refinements of the NER plan during final plan formulation and project assurance planning.

Completion of the model documentation reports (MDRs) for the model assumptions and performance overviews of the CEPP preliminary screening modeling, CEPP base condition modeling, and Alternatives 1 through 4R2 was deferred to following completion of the CEPP final array and Project Assurances/Savings Clause modeling, and the MDRs were therefore not available to be included with the Draft PIR. The CEPP MDRs were completed concurrent with the Draft PIR public review process, and the Final CEPP MDRs are included as Annex A-3 of this Appendix. Prior to the availability of the complete MDRs, RSM-BN and RSM-GL model assumption tables for Alternative 4R and Alternative 4R2 were provided in Reference 2 of the Draft PIR Hydrologic Modeling Annex A-2 to this Appendix; to maintain consistency within the Final PIR, these tables remain included in Reference 2 of the Final PIR. The following four MDR reports, which sequentially track the CEPP plan formulation and evaluation efforts, are included in Annex A-3:

- MDR 1: CEPP Model Supported Screening Efforts
- MDR 2: CEPP Baseline Runs
- MDR 3: CEPP Final Array of Alternatives (Alternatives 1 through 4)
- MDR 4: CEPP Recommended Plan (including NER refinement modeling)

The study area for the CEPP encompasses Lake Okeechobee, the Northern Estuaries (St. Lucie River and Indian River Lagoon and the Caloosahatchee River and Estuary), a portion of the EAA, the WCAs, ENP, the Southern Estuaries (Florida Bay and Biscayne Bay), and the LEC. **Section 5** of the CEPP PIR main report provides a performance evaluation for the final array of alternatives. Detailed documentation of the effects of the alternatives 1 through 4 on regional hydrology and water supply and flood control, compared to the future without project base condition, are provided in **Section 5.1.8** and **Section 5.1.15.2** and **Appendix C.2.1** of the CEPP PIR main report. Detailed documentation of the effects of the operational refinements of the Recommended Plan (Alternative 4R and Alternative 4R2) on regional hydrology and water supply and flood control, compared to the future without project base condition, are provided in Sections **5.2.8**, Section **5.2.15.2** and **Appendix C.2.2** of the CEPP PIR main report. Selected graphics are included to illustrate the performance of each alternative.

An enormous amount of output is generated from each RSM-BN and RSM-GL simulation and the accompanying post-processed performance measures. Reference maps to assist with user navigation of RSM-GL indicator regions, performance measure zones, transects, reference gauges, and viewing window spatial locations are included in Reference 3 of the Hydrologic Modeling Annex A-2. The monitoring gauge location map is provided in Figure A.8-9. Complete detailed descriptions of the RSM-BN and RSM-GL simulation output are provided in the Hydrologic Modeling Annex A-2.

The CEPP final array modeling output included two performance measure sets that include: (1) concurrent performance measure display of the CEPP FWO outputs and Alternative 1 through 4, including combined outputs for both the RSM-BN and RSM-GL models; and (2) concurrent performance measure display of the CEPP FWO outputs, Alternative 4R, and Alternative 4R2, including combined outputs for both the RSM-BN and RSM-GL models.

The CEPP PIR report documentation and two complete sets of RSM-BN and RSM-GL hydrologic model performance measure output are posted on the Everglades Plan public web site for the CERP: http://www.evergladesplan.org/pm/projects/proj_51_cepp.aspx

The following complete performance measure data sets are provided to facilitate additional review of the hydrologic modeling output for the baselines and the Recommended Plan Alternative 4R2:

- ECB, FWO, Alternative 4R, Alternative 4R2 (comparison used for NEPA evaluation in Section 5 of the main PIR report)
- ECB, 2012EC, IORBL1,Alternative 4R2 (comparison used for the Savings Clause and Project Assurances evaluation in Annex B of the PIR report)

Final CEPP hydrologic modeling products have been uploaded to the CERP Model Management System (MMS), a geographic information system (GIS) based application that includes model input data, select model output data, source code/executable files and documentation. CEPP modeling products in MMS can be accessed directly at the MMS project page through the Everglades Plan public web site: http://cerpmap1.cerpzone.org/arcgisapps/CERPMMS/CerpReport/ProjectReport.aspx?projectID=687

Sections 3.2.1 through 3.2.4 of the Hydrologic Modeling Annex A-2 provide documentation of USACE SAJ performance analysis of the hydrologic modeling results for the CEPP final array of alternatives, including operational refinements to the NER plan, with specific emphasis on engineering design considerations that were actively tracked throughout the CEPP formulation, preliminary screening, and alternative development efforts. Summary information is typically provided in this Engineering Appendix for the Recommended Plan, Alternative 4R2, only. In some cases, comparisons are additionally provided for Alternatives 4 and 4R, since Alternative 4 was the alternative selected for further optimization and subsequently refined through Alternative 4R and, ultimately the Recommended Plan, Alternative 4R2. Detailed analyses for all final array alternatives are available only in the Hydrologic Modeling Annex A-2.

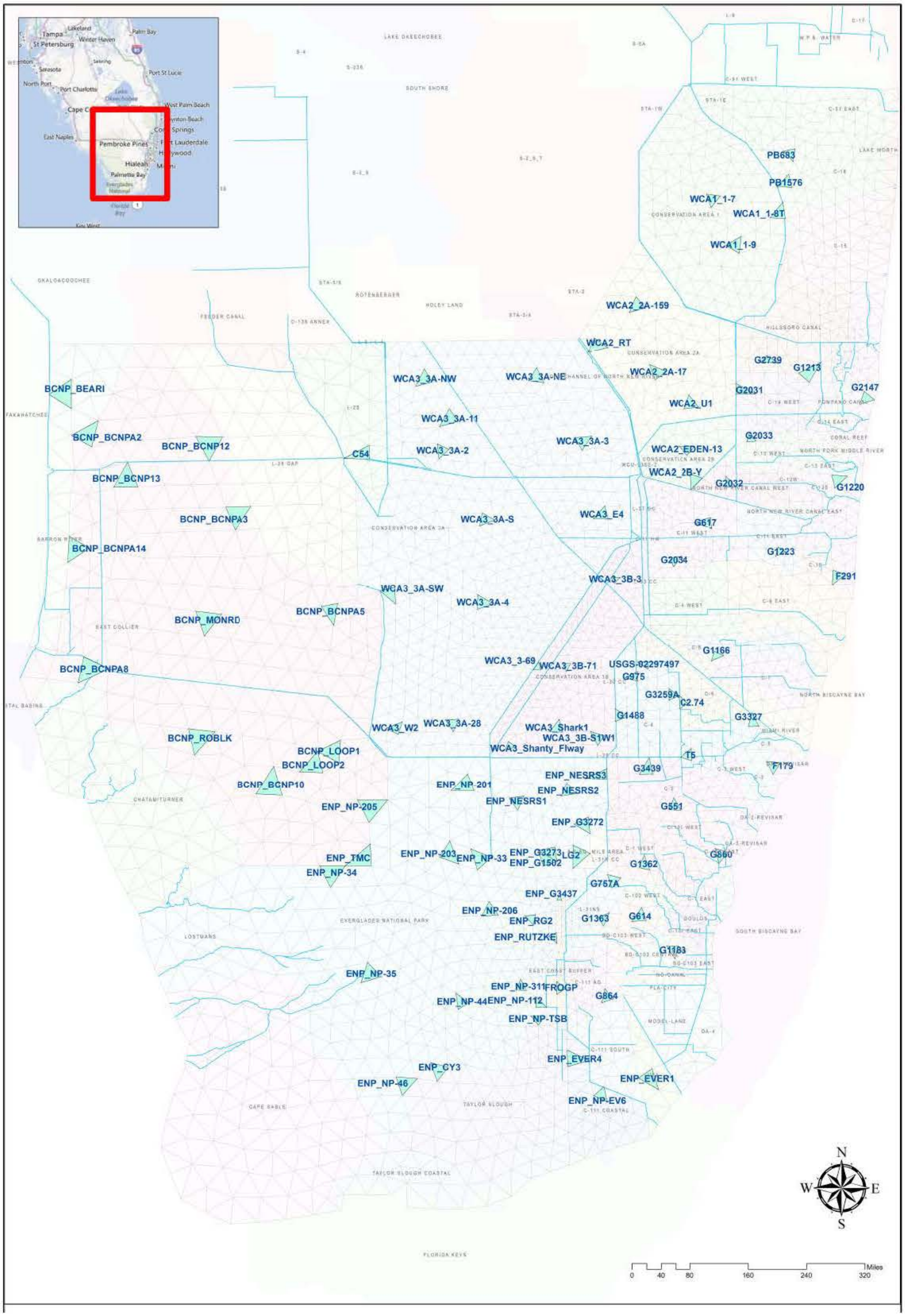


FIGURE A.8-9: MAP OF RSM-GL MONITORING GAUGE LOCATIONS

A.8.3.2.1 WCA-3A High Water Performance Criteria

The USACE Final Everglades Restoration Transition Plan (ERTP) EIS and Record of Decision (ROD signed on 19 October 2012) identified the 1960 WCA-3A 3A 9.5 to 10.5 feet, NGVD Regulation Schedule as an interim measure water management criterion for WCA-3A Zone A. This change to Zone A, compared to the previous Interim Operational Plan (IOP) for WCA-3A regulation, was necessary to mitigate for the observed effects, including discharge limitations of the S-12 spillways. The preliminary USACE Water Resources Engineering Branch (EN-W) analysis of WCA-3A high water levels, which was integrated into the ERTP EIS, also recommended further consideration of additional opportunities to reduce the duration and frequency of Water Conservation Area 3A high water events (ERTP Final EIS, Appendix A-5).

The ERTP analysis of WCA-3A high water events indicated that, based on current system conditions as simulated in the water budget spreadsheet, the IOP infrastructure and operational configuration of WCA-3A would result in a predicted increase in the Standard Project Flood (SPF) stage for WCA-3A of between 1.3 and 1.4 feet compared to the WCA-3A design assumptions (1960 General Design Memorandum (GDM), C&SF Project for Flood Control and Other Purposes, Part I, Supplement 33). Predicted SPF stages are increased from 12.40 to 13.76 feet NGVD and from 13.90 to 15.20 feet NGVD for the S-12 headwater stage and the WCA-3A three-gauge average stage, respectively. The ERTP analysis also illustrated, through the use of current USGS rating curves for the S-12 spillways, that the peak SPF stage is increased over the original design due to a reduction in outlet capacity from WCA-3A through the S-12s. This significant change to the original design assumptions, with the additional diminished extent of emergent vegetation within WCA-3A, led the USACE to identify WCA-3A high water stages as a potential cause for concern. Due to the simplistic nature (i.e., volumetric and not hydraulic routing) of the ERTP (Phase 1) analysis, the level of flood protection afforded by WCA-3A was not completely addressed during the initial assessment under ERTP; additional analyses, as identified for inclusion under a subsequent detailed study phase (termed Phase 2 in the ERTP Final EIS), are required to investigate and specify the level of protection afforded by the WCA-3A water management regime and levee configuration. A more complete documentation of the ERTP analysis, assumptions, conclusions, and recommended additional analyses is included in the Hydrologic Modeling Annex A-2.

The information on which the USACE relied on to require the ERTP WCA-3A Zone A as an interim risk reduction measure for WCA-3A high water levels has not changed prior to CEPP formulation, and no new information is currently available compared to the July 2010 assessment included as Appendix A-5 of the ERTP Final EIS. Throughout CEPP formulation, the EN-W advocated that CEPP formulation efforts attempt to maintain the frequency, duration, and peak stages of high water levels within WCA-3A consistent with the CEPP Future Without Project (FWO) condition, which includes ERTP, given recognition of the WCA-3A high water concerns identified with ERTP; prior to CEPP formulation, the January 2012 CEPP Risk Register explicitly recognized that the ERTP constraint precluded raising of the top of the WCA-3A Regulation Schedule, while simultaneously recognizing that substantial benefits were still expected and that goals to further lower stages in WCA-3A were consistent with the constraint. EN-W also indicated that it would continue to rely on the WCA-3A three-gauge average stages for assessment of WCA-3A high water frequency, durations, and peak stages, consistent with the original WCA-3A design assumptions and the ERTP assessment (average of stages at the monitoring gauges of 3A-3, 3A-4, and 3A-28); increased weight would not be considered for a single gauge, such as 3A-28 (Site 65). It was further noted that if CEPP can provide operational assurances of additional WCA-3A outlet capacity under high water conditions, including adequate consideration of potential WCA-3B seepage management and/or ecological operational limitations, the EN-W may be able to further consider proportional relaxation of the WCA-3A FWO high water duration and frequency targets.

Preliminary CEPP formulation efforts for the Green and Blue Line components, which relied on the iModel, were not able to demonstrate achievement of the FWO frequency of time within Zone A of the ERTP WCA-3A Regulation Schedule, based on the system-wide optimization of ecological targets and consideration of the additional ~220 kAF of inflows to WCA-3A available from the Flow Equalization Basin (FEB) and associated water quality treatment (refer to section 3.2.3 of the CEPP PIR main report and Appendix E.1 for additional discussion). Significant increases in WCA-3A regulatory discharge capacity were also not identified during the preliminary iModel screening.

The requirements to maintain the frequency, duration, and peak stages of high water levels within WCA-3A consistent with the CEPP Future Without Project (FWO) condition were actively integrated into the formulation efforts to identify the CEPP final array of alternatives, and the assessment of the final array demonstrated levels of performance consistent with this requirement. The EN-W assessment relied on additional post-processing of the RSM-GL model results, as subsequently discussed.

To establish the WCA-3A high water performance criteria to assist with CEPP formulation and to provide technical recommendations to the CEPP formulation efforts, EN-W developed comparisons between the ERTP Recommended Plan modeling (Alternative 9E1 in the ERTP Final EIS), which was simulated with the SFWMM, and the RSM-GL base conditions representations that were developed for CEPP starting in May 2012. Based on the results of these comparisons, EN-W recommended in July 2012 that CEPP formulation efforts should identify alternative configurations which maintain the frequency, duration, and peak stages of high water levels within WCA-3A consistent with the CEPP FWO condition. Additional details and results of this comparison are provided in the Hydrologic Modeling Annex A-2.

Compared to the CEPP FWO (final December 2012 release), the CEPP alternative 4R2 stages are lowered by approximately 0.1-0.3 feet in the upper 10 percent of the stage duration curve for the WCA-3A three-gauge average stage, as shown in Figure A.8-10 (full stage duration curve) and Figure A.8-11 (upper 25 percent of the stage duration curve). In order to consider potential differences during specific years, the EN-W assessment also considered the annual duration of exceedance of the ERTP WCA-3A Zone A stage levels for the complete period of simulation (Figure A.8-12). The annual durations were also displayed and assessed as a frequency curve (Figure A.8-13). The total number of days above Zone A is summarized as follows for the CEPP FWO and CEPP alternatives (with percent of total period of simulation, 14975 days, in parentheses): CEPP FWO – 2718 days (18.15%); Alternative 1 – 3206 days (21.41%); Alternative 2 – 3034 days (20.26%); Alternative 3 – 3285 days (21.94%); Alternative 4 – 3227 days (21.55%); Alternative 4R – 3307 days (22.08%); and Alternative 4R2 – 3323 days (22.19%).

The EN-W performance assessment for the final array of alternatives also included review of the WCA-3A stage hydrographs for individual years in which the number of days above Zone A increased by more than 20 percent between the CEPP FWO and any of the CEPP alternatives, as shown highlighted in Table A.8-1. In the Hydrologic Modeling Annex A-2, annual hydrographs are provided for each of the twelve years which triggered this further detailed assessment (Figures 25 through 38 in Annex A-2): 1969, 1980, 1983-1985, 1993-1996, 1999, 2003, and 2005.

Annual stage hydrograph statistical distribution plots were developed to assist with the general characterization of differences in intra-annual stage variability, to facilitate comparisons between the CEPP FWO baseline condition (Figure A.8-14) and CEPP Alternative 4R2 (Figure A.8-15). For the 41-year period of simulation, the graphics illustrate the maximum and minimum stage, 90th and 10th percentile stages, 75th and 25th percentile stages, median stage, and mean stage at a daily time step. The graphics also include the ERTP WCA-3A Regulation Schedule Zone A reference line, the FWS MSTS recommended seasonal range for January 1 and May 1-31, and the average ground surface elevation (GSE) for the WCA-3A 3-gauge average at 8.34 feet NGVD (3A-3 GSE 9.08 feet NGVD; 3A-4 GSE 8.49 feet NGVD; 3A-28 GSE 7.44 feet NGVD). Compared to the CEPP FWO, the following general trends are noted for Alternative 4R2: increased stages through the dry season, particularly January through May (most evident for the 75th and 90th percentiles); increased stages at the end of the dry season in May (most evident for 10th through 90th percentiles); increased stages at the beginning of the wet season in June-July (evident under all conditions); increased stages through the wet season and start of the dry season during average to dry years (evident for minimum to median stages); reduced stages at the end of the wet season in September-October during wet years (90th percentile and maximum stage); and reduced stages at the beginning of the dry season in November and December during wet years (90th percentile and maximum stages). These graphics illustrate that the increased durations within Zone A with the CEPP alternatives, as compared to the CEPP FWO, are the combined result of higher stages at the end of the dry season (along the Zone A recession) and higher antecedent stages at the beginning of the wet season (June 1) with the resulting effects of early wet season rainfall events. Peak stages and durations of Zone A exceedance at the end of the wet season, when WCA-3A design limitations are most critical due to the maximum stages, do not increase and, in many instances, are significantly reduced compared to the FWO condition. This conclusion is consistent with detailed review of the annual hydrographs presented in the Hydrologic Modeling Annex A-2.

The detailed EN-W assessment of the frequency, duration, and peak stages of high water levels within WCA-3A concluded: (1) WCA-3A peak stages are lowered (these stages are most critical for WCA-3A design limitations); (2) the frequency and durations of Zone A exceedance are increased; (3) the increased frequency and durations occur during periods of the year when WCA-3A water levels are below peak critical levels; (4) CEPP infrastructure modifications (increased WCA-3A outlet capacity) and operations demonstrate that increased WCA-3A stages at the end of the dry season and start of the wet season can be effectively managed to avoid exacerbating high water conditions at the end of the wet season when Zone A levels off at 10.5 feet NGVD; and (5) CEPP infrastructure and operations utilized to achieve these performance levels need to be codified in the CEPP Project Operating Manual (POM). The requirements to maintain the frequency, duration, and peak stages of high water levels within WCA-3A consistent with the CEPP FWO were, therefore, successfully achieved based on EN-W assessment of the overall performance of the CEPP final array, including the Recommended Plan.

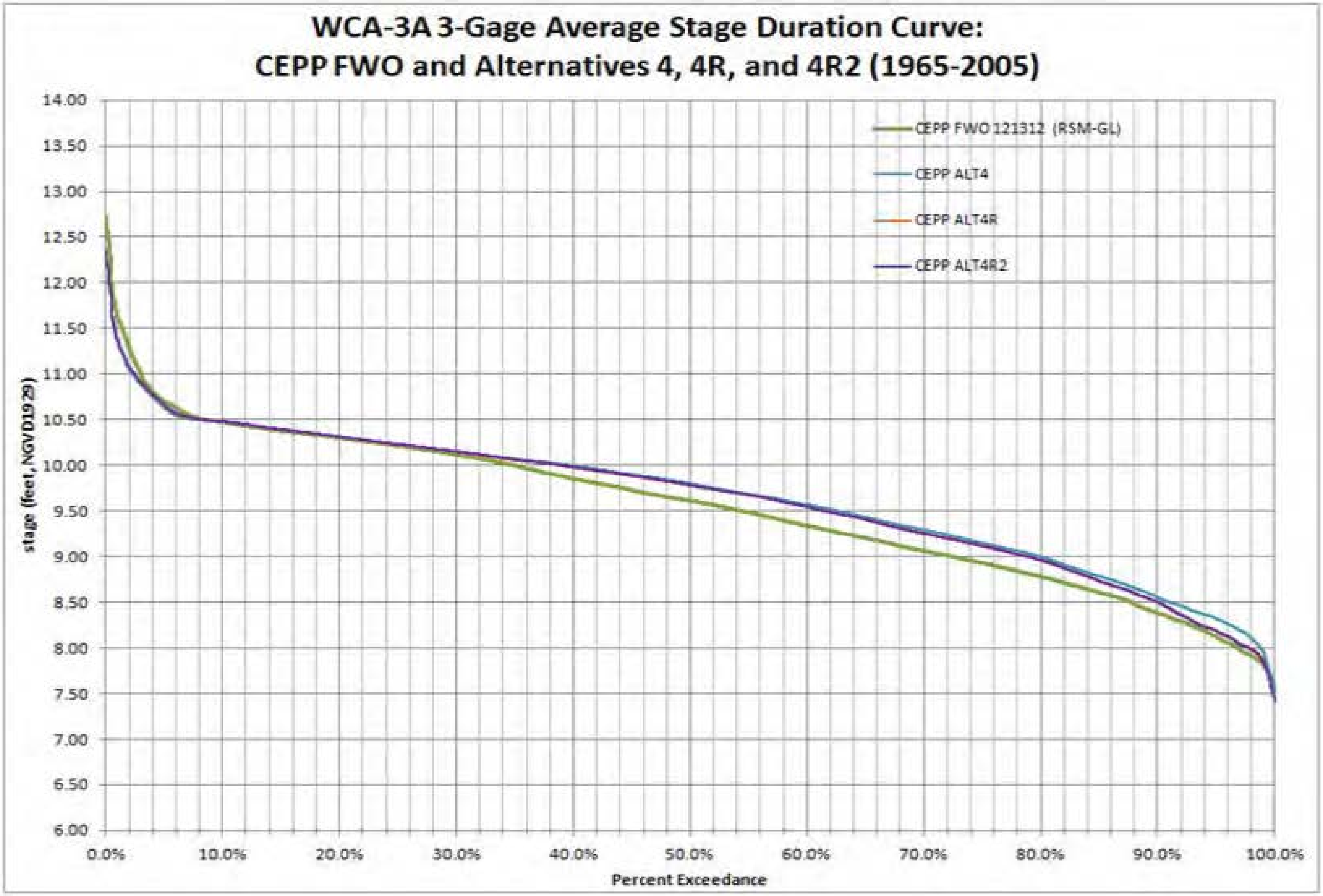


FIGURE A.8-10: WCA-3A 3-GAUGE AVERAGE STAGE DURATION CURVE FOR CEPP FWO AND CEPP ALTERNATIVES 4, 4R, AND 4R2

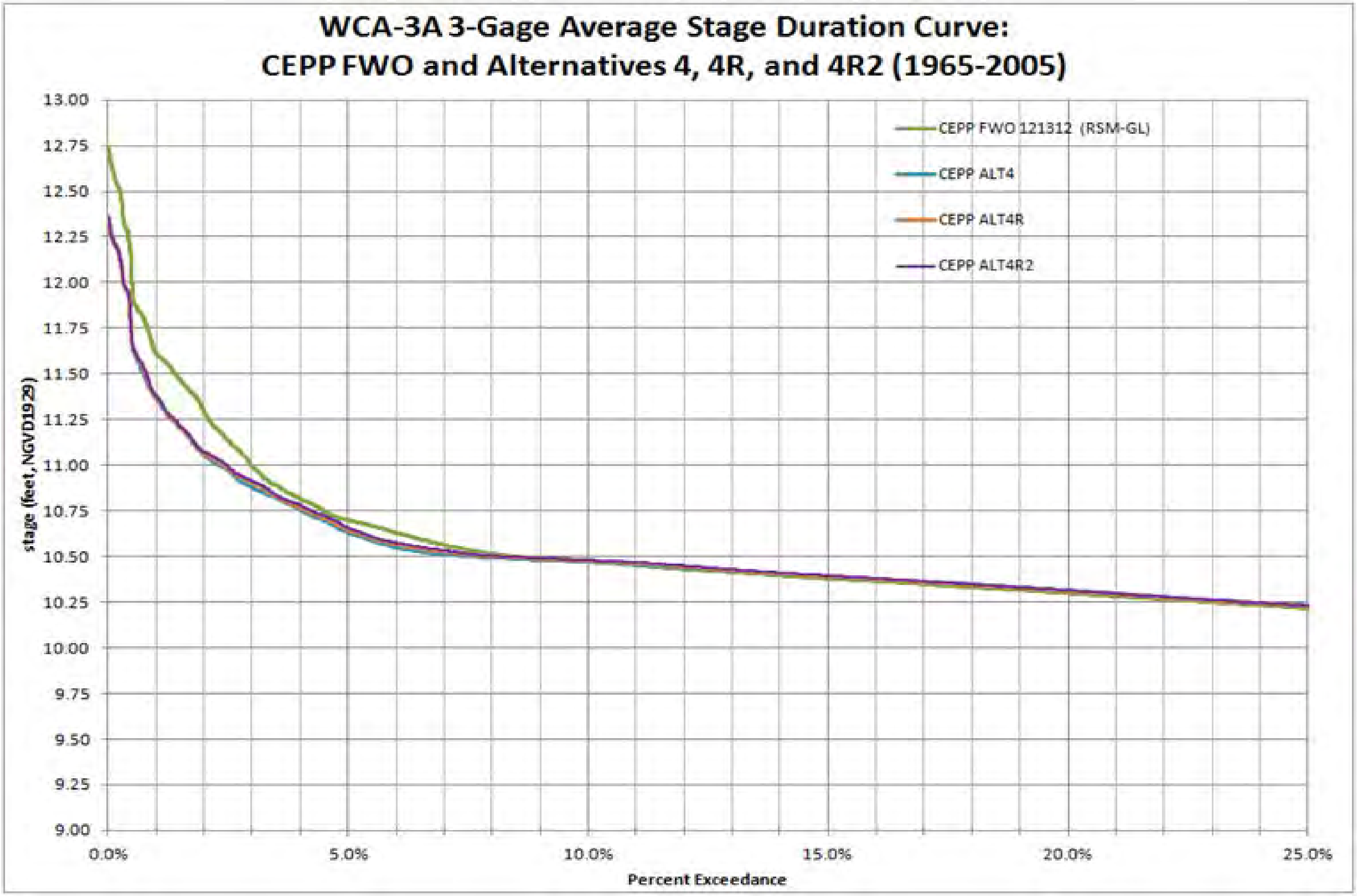


FIGURE A.8-11: WCA-3A 3-GAUGE AVERAGE STAGE DURATION CURVE FOR CEPP FWO AND CEPP ALTERNATIVES 4, 4R, AND 4R2 (UPPER 25%)

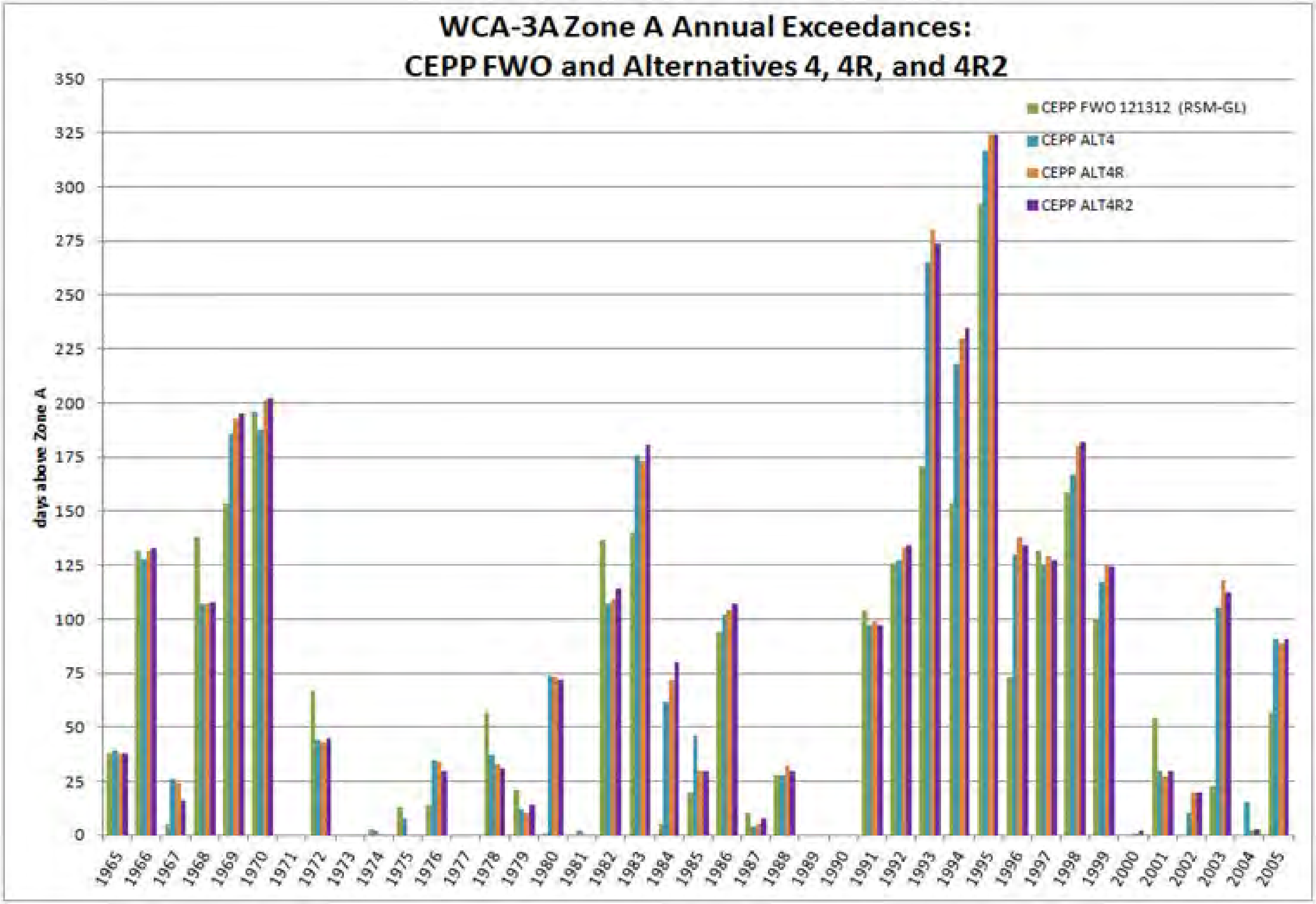


FIGURE A.8-12: WCA-3A 3-GAUGE AVERAGE ANNUAL ZONE A EXCEEDANCE FOR CEPP FWO AND CEPP ALTERNATIVES 4, 4R, AND 4R2

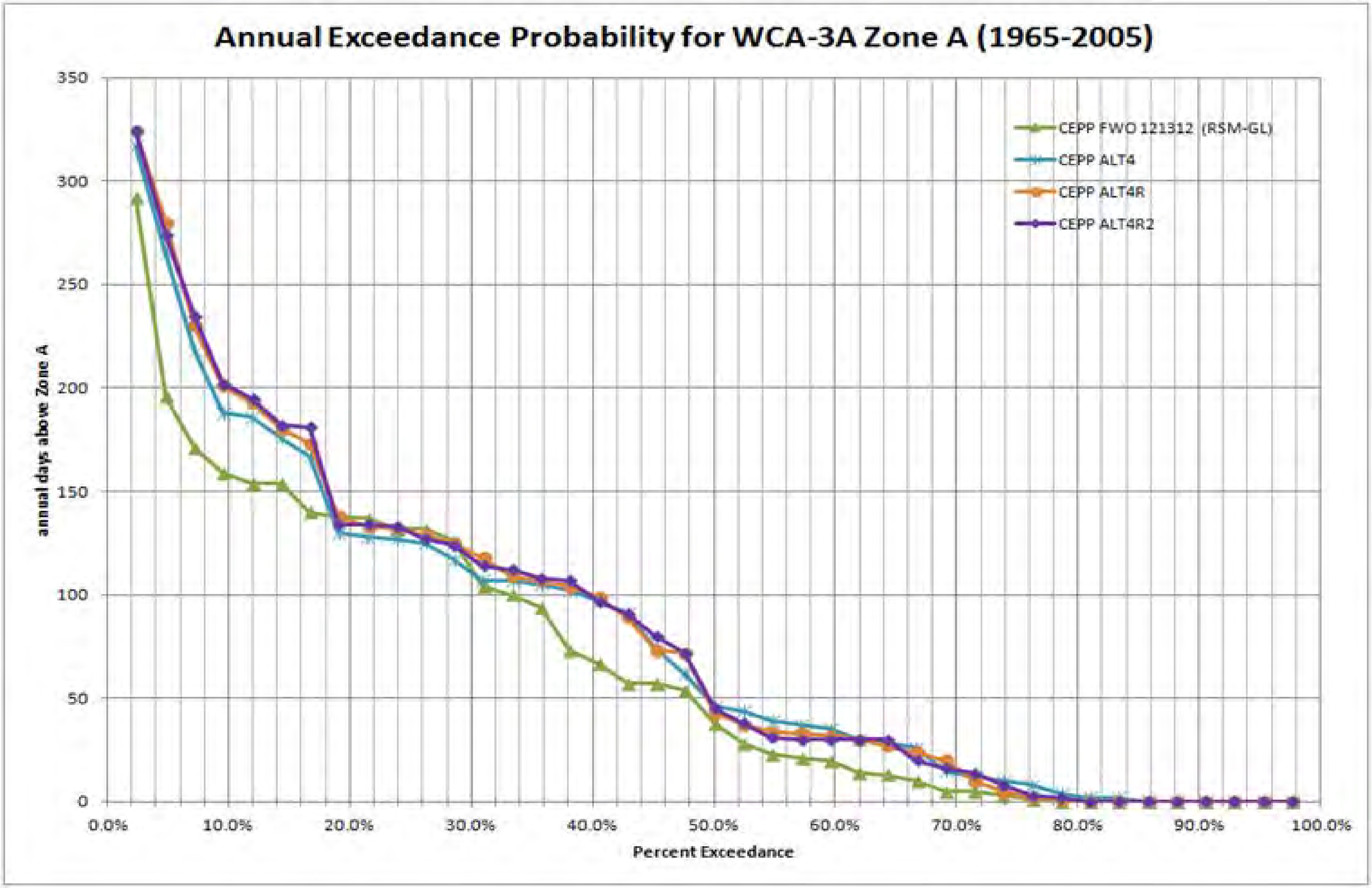


FIGURE A.8-13: WCA-3A 3-GAUGE AVERAGE PROBABILITY EXCEEDANCE PLOT FOR ANNUAL ZONE A EXCEEDANCE FOR FWO AND CEPP ALTERNATIVES 4, 4R, AND 4R2

TABLE A.8-1: ANNUAL ZONE A EXCEEDANCE DAYS (WCA-3A 3-GAUGE AVERAGE) FOR FWO AND CEPP ALTERNATIVES 4, 4R, AND 4R2

	Summary Table: WCA-3A Zone A Annual Exceedance Duration (days)			
Year	CEPP FWO 121312	CEPP ALT4	CEPP ALT4R	CEPP ALT4R2
1965	38	39	37	38
1966	132	128	132	133
1967	5	26	24	16
1968	138	107	107	108
1969	154	186	193	195
1970	196	188	201	202
1971	0	0	0	0
1972	67	44	43	45
1973	0	0	0	0
1974	3	2	0	0
1975	13	8	0	0
1976	14	35	34	30
1977	0	0	0	0
1978	57	37	33	31
1979	21	12	10	14
1980	1	74	73	72
1981	0	2	0	0
1982	137	107	109	114
1983	140	176	173	181
1984	5	62	72	80
1985	20	46	30	30
1986	94	102	104	107
1987	10	4	5	8
1988	28	28	32	30
1989	0	0	0	0
1990	0	0	0	0
1991	104	97	99	97
1992	126	127	133	134
1993	171	265	280	274
1994	154	218	230	235
1995	292	317	324	324
1996	73	130	138	134
1997	132	125	129	127
1998	159	167	180	182
1999	100	117	125	124
2000	0	0	1	2
2001	54	30	27	30
2002	0	10	20	20
2003	23	105	118	112
2004	0	15	2	3
2005	57	91	89	91
	Summary Table: WCA-3A Zone A Annual Exceedance Duration			
	CEPP FWO 121312	CEPP ALT4	CEPP ALT4R	CEPP ALT4R2
total (1965-2005 POR; 14975 days)	2718	3227	3307	3323
total (percent of POR)	18.15	21.55	22.08	22.19
percent increase vs FWO	--	18.73	21.67	22.26

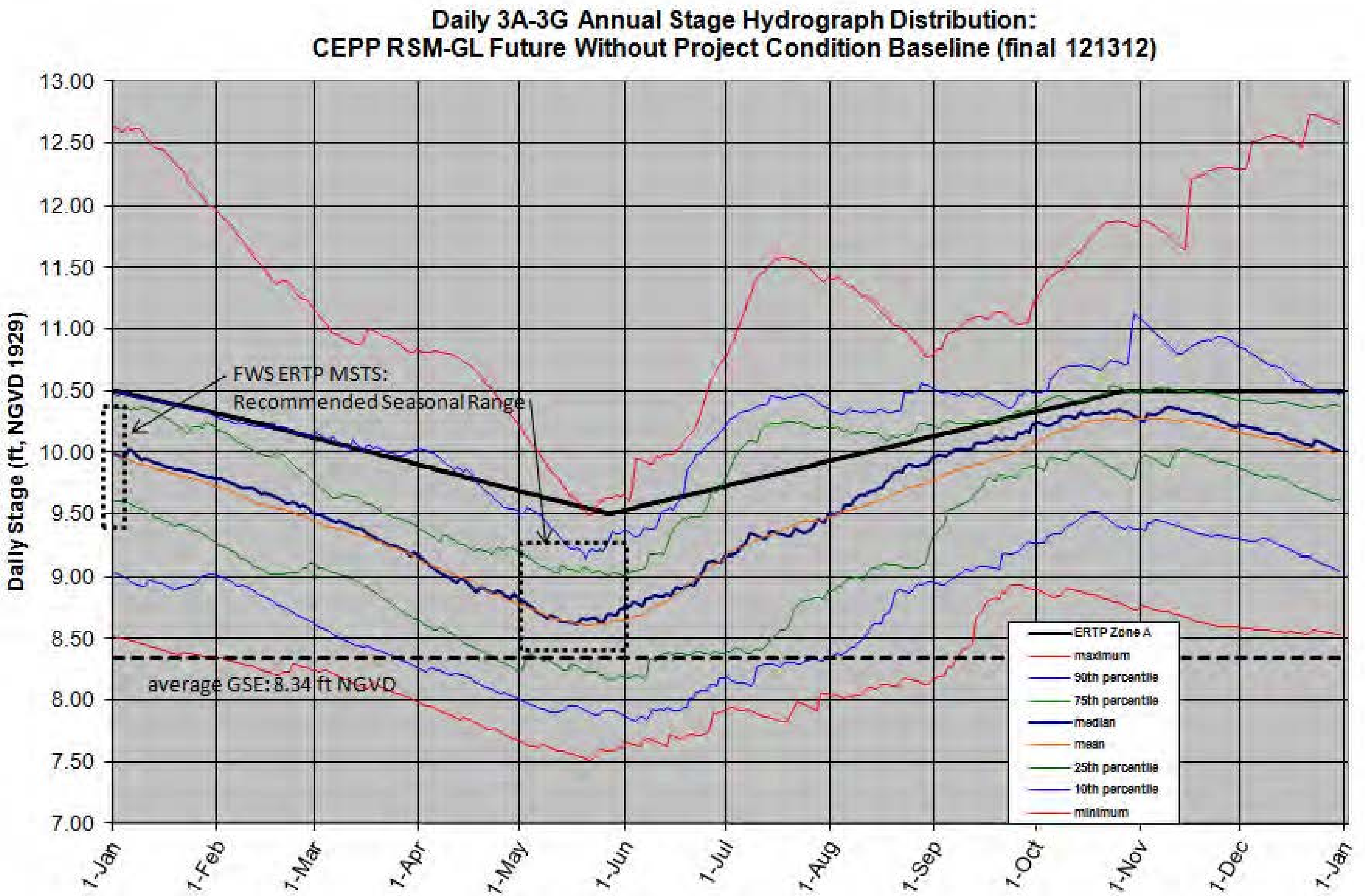


FIGURE A.8-14: WCA-3A 3-GAUGE AVERAGE ANNUAL AVERAGE STAGE HYDROGRAPHS FOR CEPP FWO

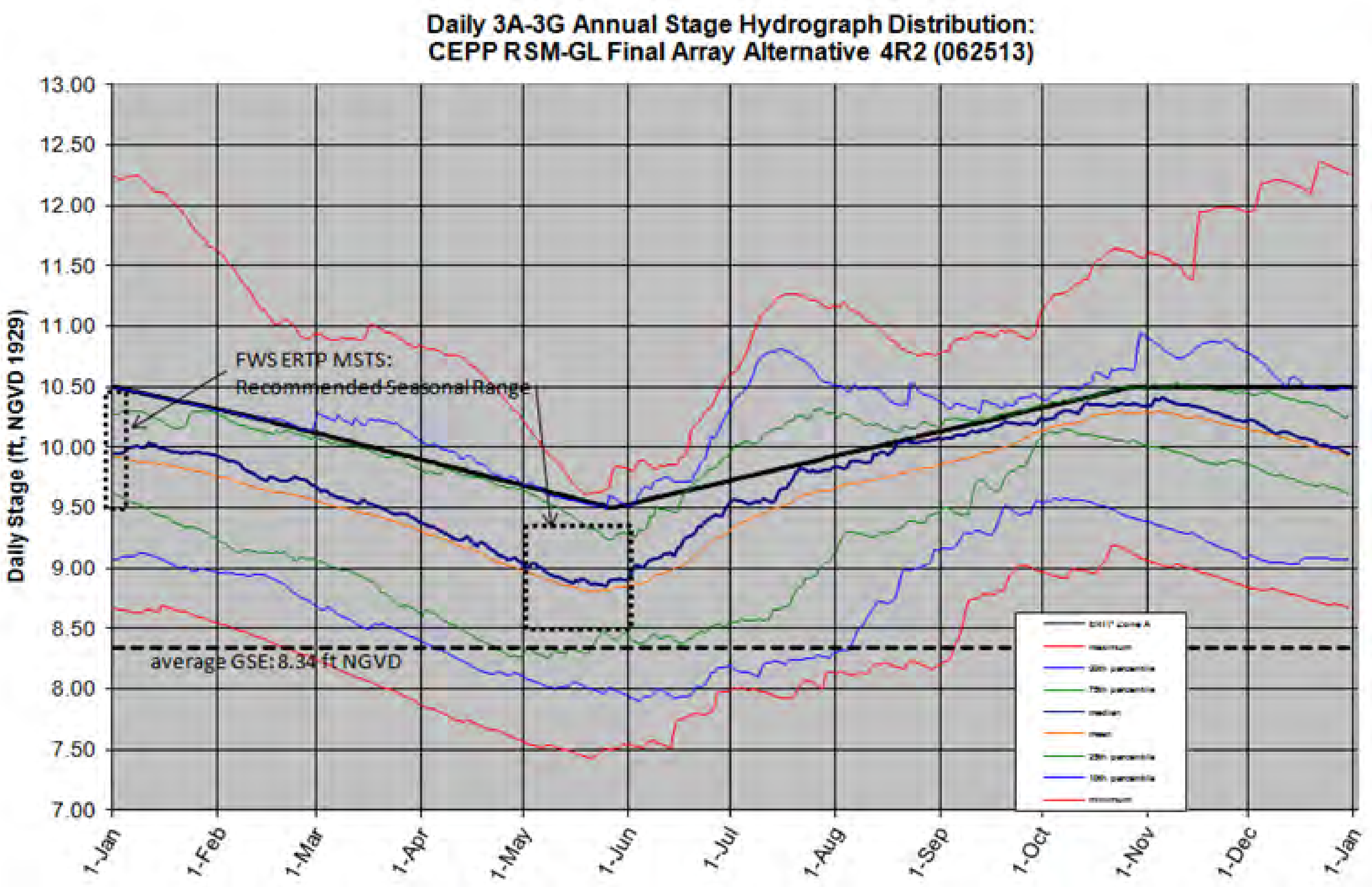


FIGURE A.8-15: WCA-3A 3-GAUGE AVERAGE ANNUAL AVERAGE STAGE HYDROGRAPHS FOR CEPP ALTERNATIVE 4R2

A.8.3.2.2 WCA-3B Design Considerations

Subsequent to completion of the L-67A Levee in 1962 (the adjacent L-67C Levee was completed in 1966), WCA-3B water levels have been highly managed. The S-151 gated culvert (1105 cfs design capacity) currently provides the only structural connection between WCA-3A and WCA-3B. The SPF stage for WCA-3B, based on Site 71 (refer to the Figure A.8-9 map), was initially established in the 1960 GDM for WCA-3 (C&SF Part 1, Supplement 33) at 8.50 feet NGVD based on an assumed 5-day, 16.5-inch rainfall event; detailed SPF flood routing information for WCA-3B is not provided in the GDM. Starting in 1985, the C&SF Experimental Program for Water Deliveries to ENP established S-151 operational criteria that discontinued S-151 regulatory releases from WCA-3A if stages at Site 71 exceed 8.5 feet NGVD. The Site 71 constraint at 8.5 feet NGVD was also used for the 1994-1995 L-67 gap tests, which were conducted as design tests for the MWD to ENP Project. The IOP and ERTP WCA-3A Regulation Schedules specify operation of S-151 for water supply only during Column 1 operations (no WCA-3A regulatory releases to the South Dade Conveyance System (SDCS)) and S-151 regulatory inflows to WCA-3B during Column 2 operations (WCA-3A regulatory releases to the SDCS), contingent on the Site 71 stage being below 8.5 feet NGVD.

The USACE has not conducted a comprehensive review of the previously-established SPF stages for WCA-3B, pending consideration of modified inflow infrastructure for WCA-3B. SFWMM modeling conducted for the 1993 MWD to ENP Feature Design Memorandum (FDM), based on the 1992 MWD GDM default operational plan, identified a revised SPF stage of 11.6 feet NGVD at Site 71 for the MWD Project condition; however, despite subsequent multiple interagency efforts, a final configuration for WCA-3B inflow structures and an associated MWD operational plan, has not been identified prior to the conclusion of CEPP formulation efforts.

Concurrent with CEPP alternative formulation and modeling efforts, EN-W conducted a review of WCA-3B high water levels compared to the WCA-3B design criteria and independent of any previous SPF stage considerations. WCA-3B is currently bounded by the L-29 Levee (Section 3) to the south, the L-67A Levee and the L-67C Levee to the west, and the L-30 Levee to the east; the design grades for these WCA-3B perimeter levees range between 13.0 feet NGVD for the L-29 Levee (note: typical sections range from 13.5-17.5 feet NGVD, due to subsequent stockpiling of spoil material from L-29 Canal improvements, and all L-29 Section 3 Levee sections meet or exceed the design grade) to 20.0 feet NGVD for the L-30 Levee (the design grades for the L-67A and L-67C Levees are 17.5 and 12.5 feet NGVD, respectively), such that the L-29 Levee design grade represents the limiting factor for peak WCA-3B stages for CEPP. Stage duration curves (upper 25%) for the CEPP ECB, CEPP FWO, Alternative 4, Alternative 4R, and Alternative 4R2 are provided in Figures A.8-16 and A.8-17 for the two RSM-GL monitoring gauge locations within WCA-3B at Site 71 and Shark-1 (also alternatively referred to as SRS-1) that are produced with the model standard output information; corresponding RSM-GL model GSE elevations for these gauges are 6.64 and 6.61 feet NGVD, respectively. For CEPP alternative 4R2, peak stages within WCA-3B (outside of the Blue Shanty Flow-way in Alternative 4R2) were 9.25 and 9.24 feet NGVD at Site 71 and Shark-1, respectively, or approximately 0.20 feet greater than the CEPP ECB and CEPP FWO baselines (9.05-9.06 feet NGVD); however, the WCA-3B peak stages for the CEPP Recommended Plan remains approximately 3.75 feet below the L-29 Section 3 design grade of 13.0 feet NGVD. The SPF rainfall for WCA-3B is approximately 1.5 feet (17.5 inches; based on the localized 3-day, 100-year maximum rainfall event of 14 inches). Based on EN-W assessment of these WCA-3B peak water depths less than 3 feet (2.61-2.63 feet peak depth for Alternative 4R2 stages), maximum wind and wave run-up potentials would not be expected to exceed 1-2 feet. For this preliminary EN-W assessment (further analysis will be conducted during PED), a presumed worst-case scenario was defined for the CEPP Recommended Plan, with peak Alternative 4R2 stages exacerbated by the additional SPF rainfall and maximum wind and wave run-up depths. Under this assumed worst-case scenario (9.25 feet NGVD stage + 1.5 feet SPF rainfall + 2.0 feet run-up potential), the L-29 Section 3 Levee would not be expected to be overtopped at the two lowest elevation points (with approximately 0.25 feet of remaining freeboard, compared to the minimum L29 Section 3 Levee elevation of 13.0 feet NGVD). Given no predicted L-29 Section 3 Levee overtopping for this conservative assumed combination of events and recognition that CEPP inflows to WCA-3B (both within the Blue Shanty flow-way and eastern WCA-3B) will utilize controllable structures that may be closed in anticipation of extreme rainfall events, the EN-W preliminary assessment of the WCA-3B design criteria concluded that the proposed CEPP water levels of Alternative 4R2 would not adversely affect the flood control capability of the unmodified eastern segment of the L-29 Levee (or other perimeter levees, which have higher design elevations) bordering WCA-3B. The USACE currently anticipates revisiting the WCA-3B SPF stage during PED, pending final authorization of the CEPP and the establishment of operating criteria for WCA-3B water management structures for a System Operating Manual revision for CEPP implementation.

Maximum stages within the WCA-3B Blue Shanty flow-way and maximum head differential across the L-67D Levee are utilized for the hydraulic, geotechnical, and civil design of the L-67D Levee for the CEPP Recommended Plan, Alternative 4R2. Stage duration curves for the interior of the Blue Shanty flow-way, external to the flow-way at the Shark 1 gauge in WCA-3B, and within the L-29 Canal, both west of the CEPP-proposed S-355W L-29 gated spillway structure (within the flow-way, following CEPP removal of this section of the L-29 Levee) and east of the S-355W structure, are shown in Figure A.8-18 for Alternative 4R2. The head differential across the L-67D Levee for the CEPP Recommended Plan is shown in Figure A.8-19 and Figure A.8-20 in both time series format and frequency curve format. The maximum head differential across the CEPP-proposed L-67D Levee is approximately 1.50 feet during the 1965-2005 RSM-GL period of simulation.

For additional reference, the L-29 Canal stage duration curves for the ECB, FWO, Alternative 4, Alternative 4R, and Alternative 4R2 are shown in Figure A.8-21 and Figure A.8-22 (stages correspond to the western reach of the L-29 Canal for Alternatives 4, 4R, and 4R2, west of the S-355W structure). Peak L-29 Canal stages for CEPP will need to be considered for future implementation of the DOI TTNS roadway modifications, including the potential need to further raise the eastern portion of the Tamiami Trail roadway, east of the CEPP-proposed S-355W L-29 gated spillway structure. Peak simulated L-29 Canal stages for Alternative 4R2 are 9.59 feet NGVD west of the S-355W structure and 9.50 feet NGVD east of the S-355W structure (refer to Figure A.8-18).

Annual stage hydrograph statistical distribution plots for the L-29 Canal across the 1965-2005 period of simulation are provided for the IORBL1 updated future without project condition baseline and Alternative 4R2 in Figure A.8-23, Figure A.8-24, and Figure A.8-25. Since Alternative 4R2 includes the proposed S-355W structure within the L-29 Canal, the L-29 Canal statistical plots are separately reported for the West L-29 Canal in Figure A.8-24 (west of the S-355W structure) and the East L-29 Canal in Figure A.8-25 (east of the S-355W structure). The mean daily L-29 stage hydrograph, maximum daily L-29 stage hydrograph, and maximum daily L-29 stage hydrograph for the 2012Ec updated Existing Condition Baseline, IORBL1 and Alternative 4R2 are comparatively shown in Figure A.8-26, Figure A.8-27, and Figure A.8-28, respectively. These additional graphics have been included in the Final PIR to demonstrate the effects of the CEPP on the L-29 Canal, between the southern boundary of WCA 3B and the northern boundary of the ENP NESRS, for the complete range of hydrologic condition included in the period of simulation (1965-2005).

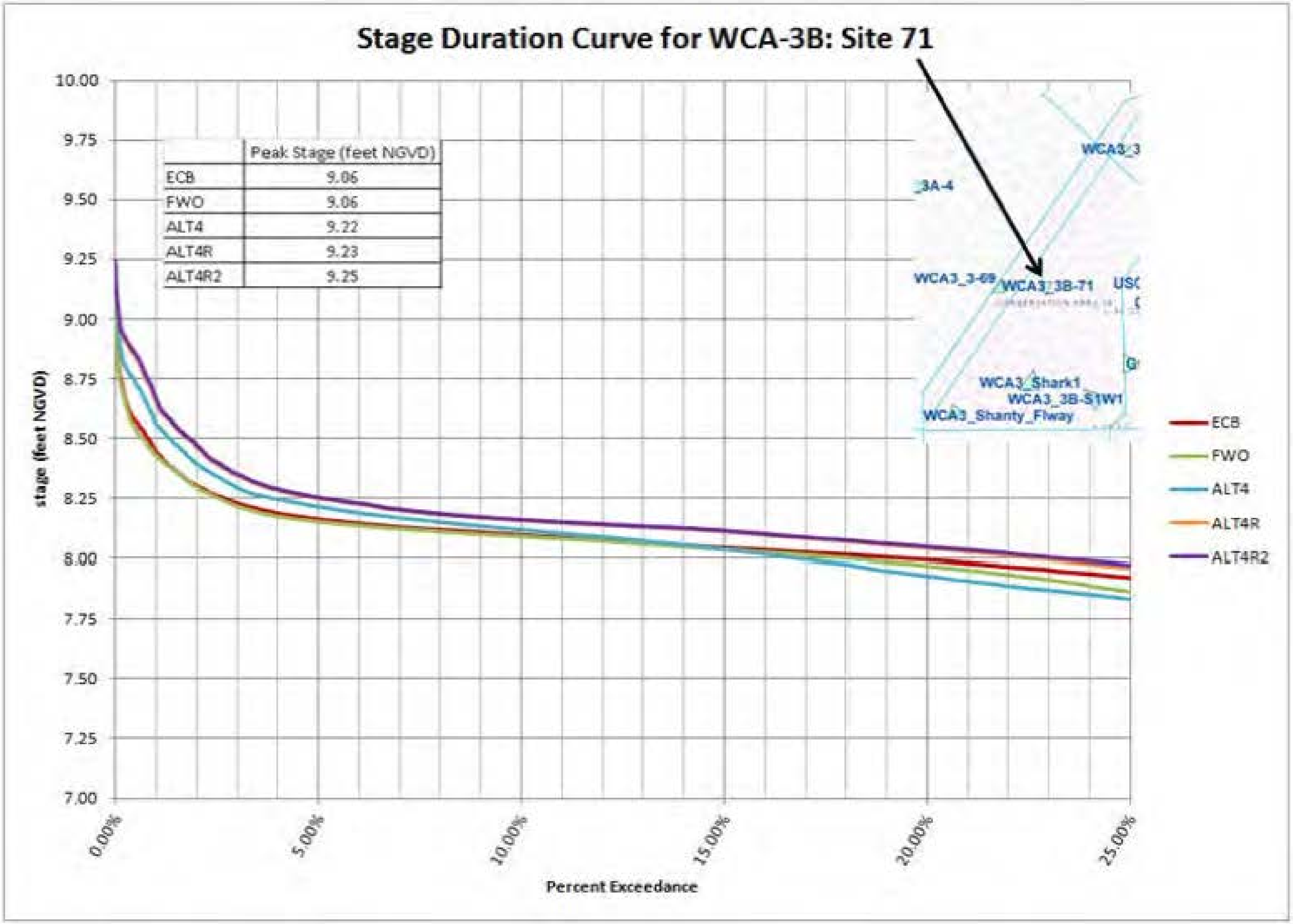
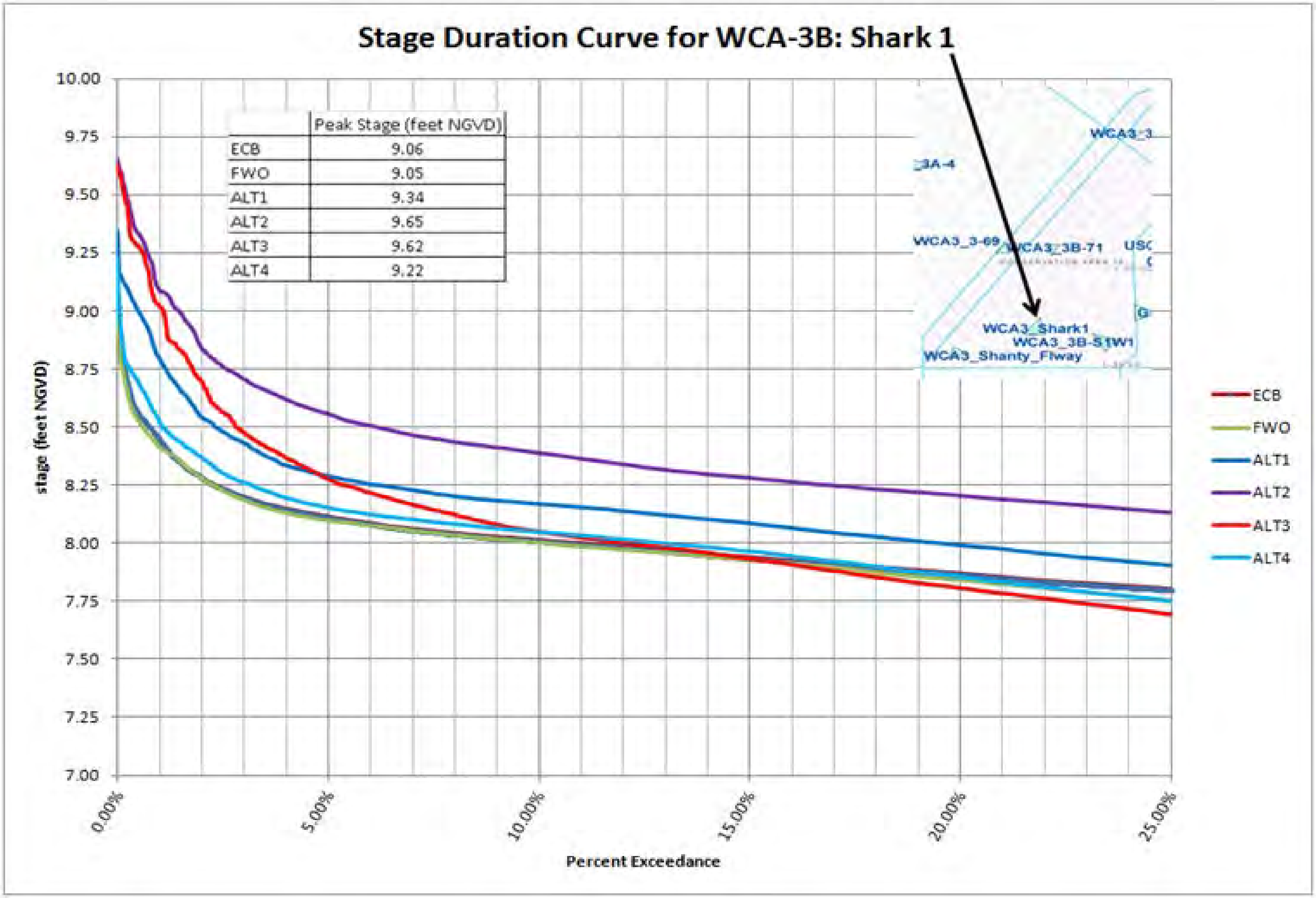


FIGURE A.8-16: WCA-3B SITE 71 STAGE DURATION CURVES FOR CEPP BASELINES AND CEPP ALTERNATIVES 4, 4R, AND 4R2 (UPPER 25%)



	Peak Stage (feet NGVD)
ECB	9.06
FWO	9.05
ALT1	9.34
ALT2	9.65
ALT3	9.62
ALT4	9.22

FIGURE A.8-17: WCA-3B SHARK-1 STAGE DURATION CURVES FOR CEPP BASELINES AND CEPP ALTERNATIVES 4, 4R, AND 4R2 (UPPER 25%)

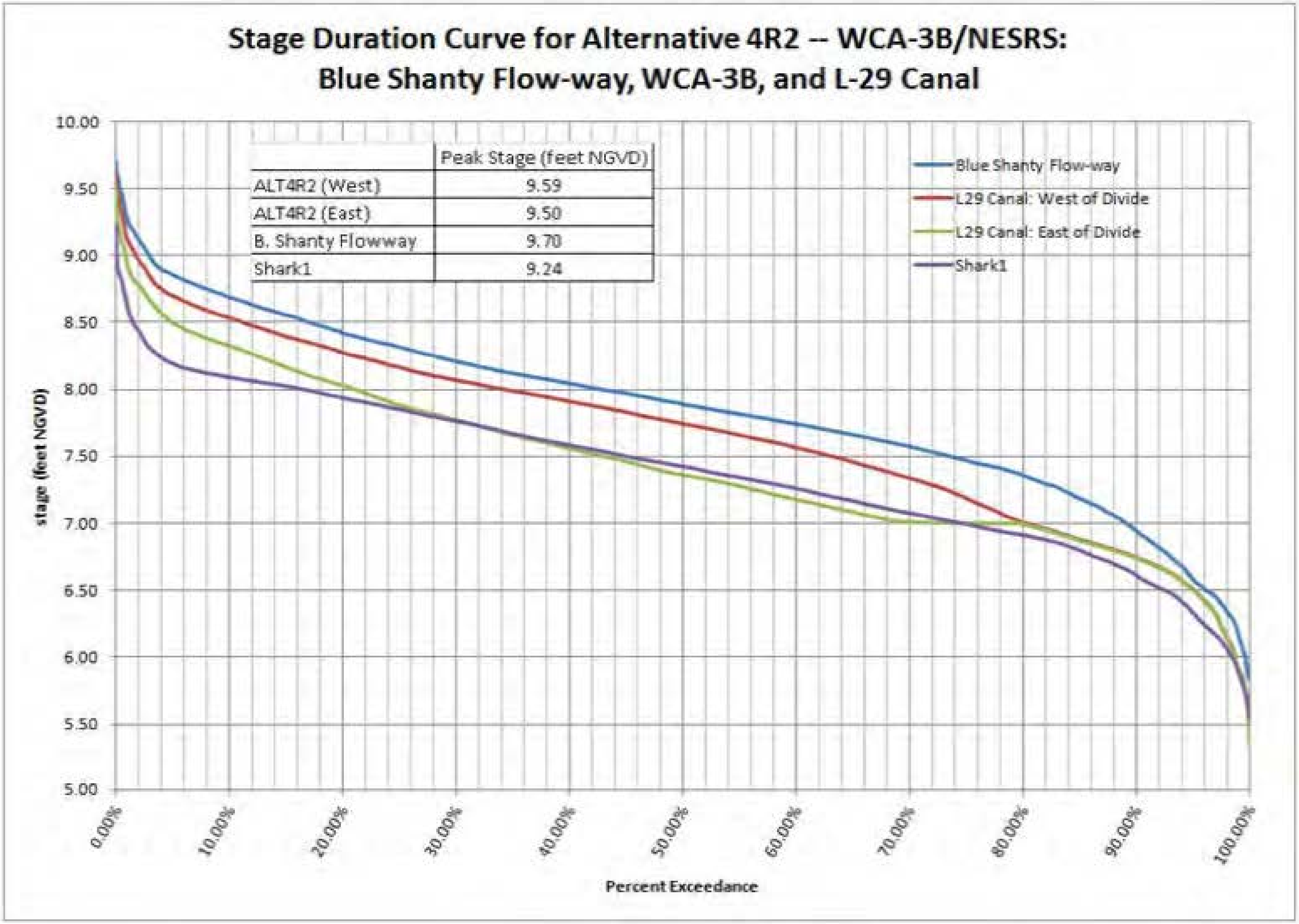


FIGURE A.8-18: L-29 CANAL AND BLUE SHANTY FLOW-WAY STAGE DURATION CURVES FOR CEPP ALTERNATIVE 4R2

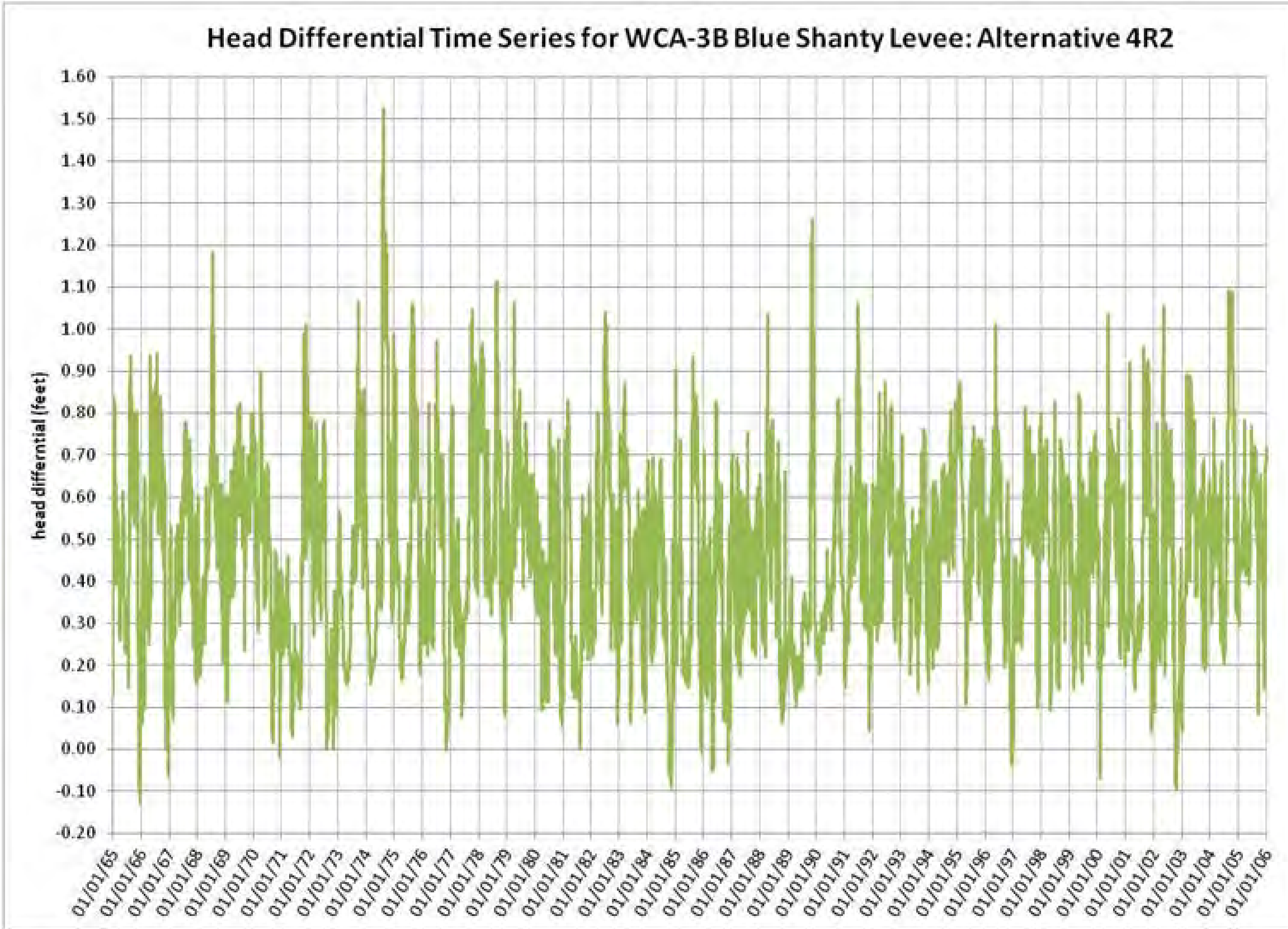


FIGURE A.8-19: L-67D HEAD DIFFERENTIAL TIME SERIES FOR CEPP ALTERNATIVE 4R2

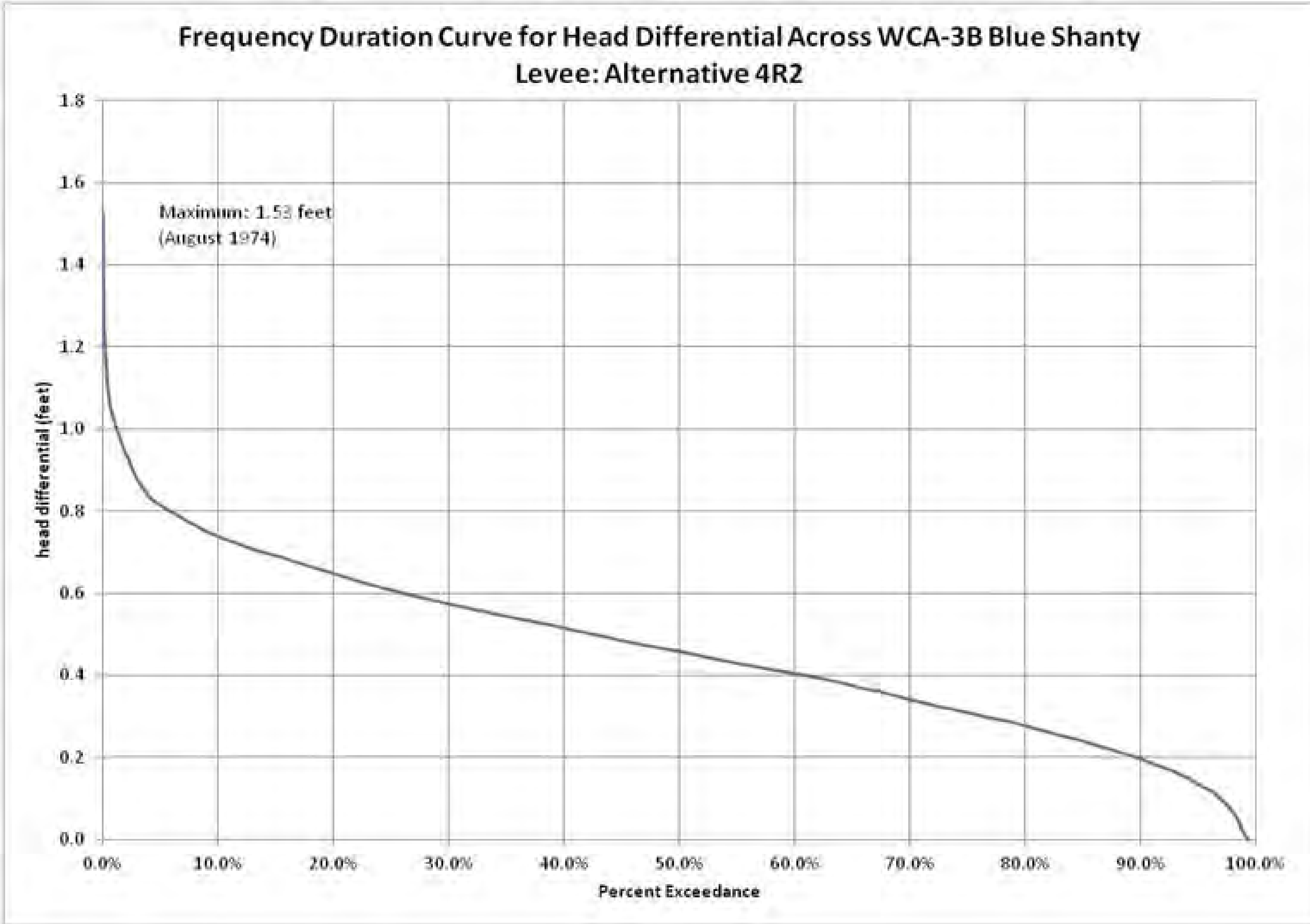


FIGURE A.8-20: L-67D HEAD DIFFERENTIAL FREQUENCY CURVE FOR CEPP ALTERNATIVE 4R2

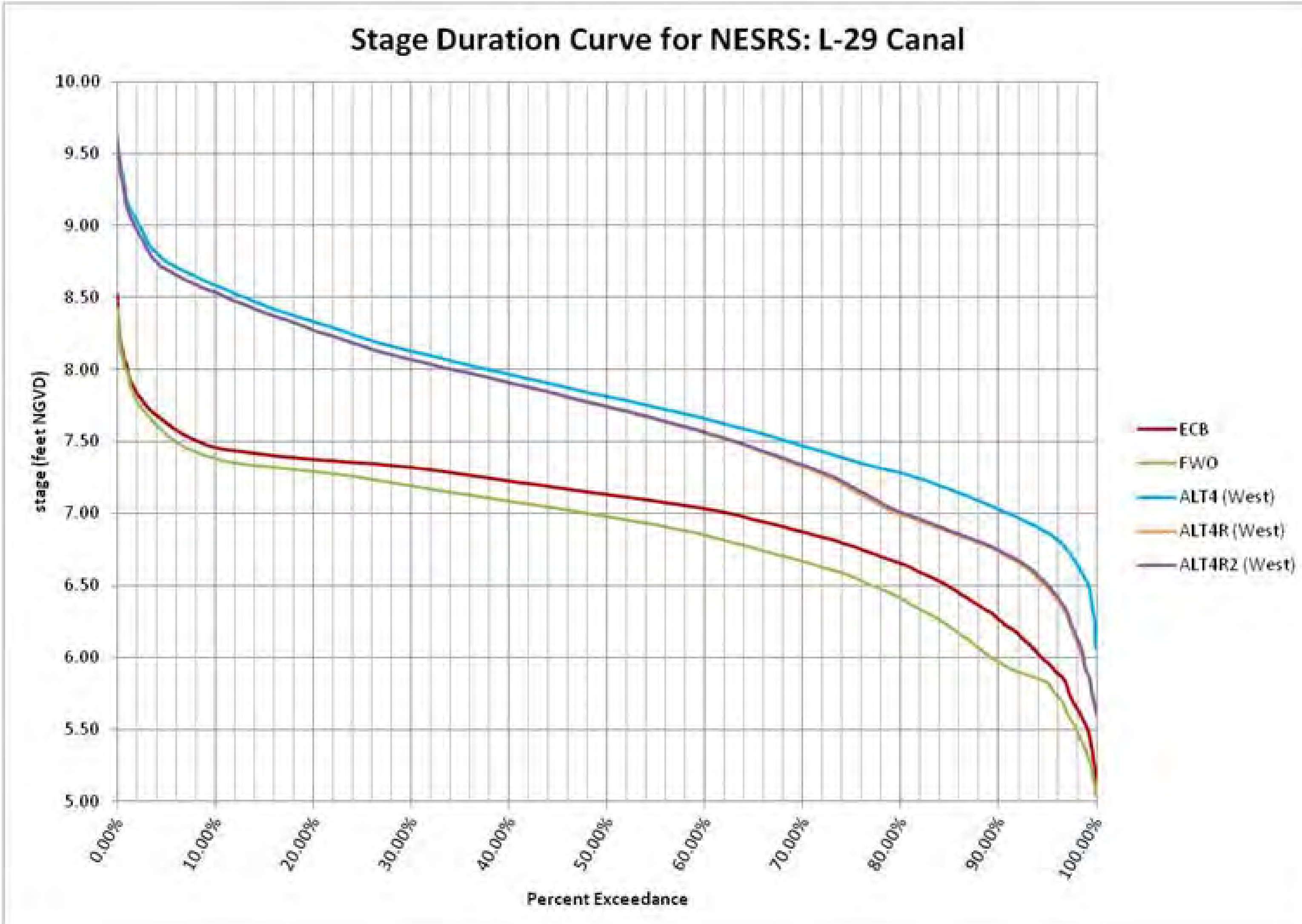


FIGURE A.8-21: L-29 CANAL STAGE DURATION CURVES FOR CEPP BASELINES AND CEPP ALTERNATIVES 4, 4R, AND 4R2

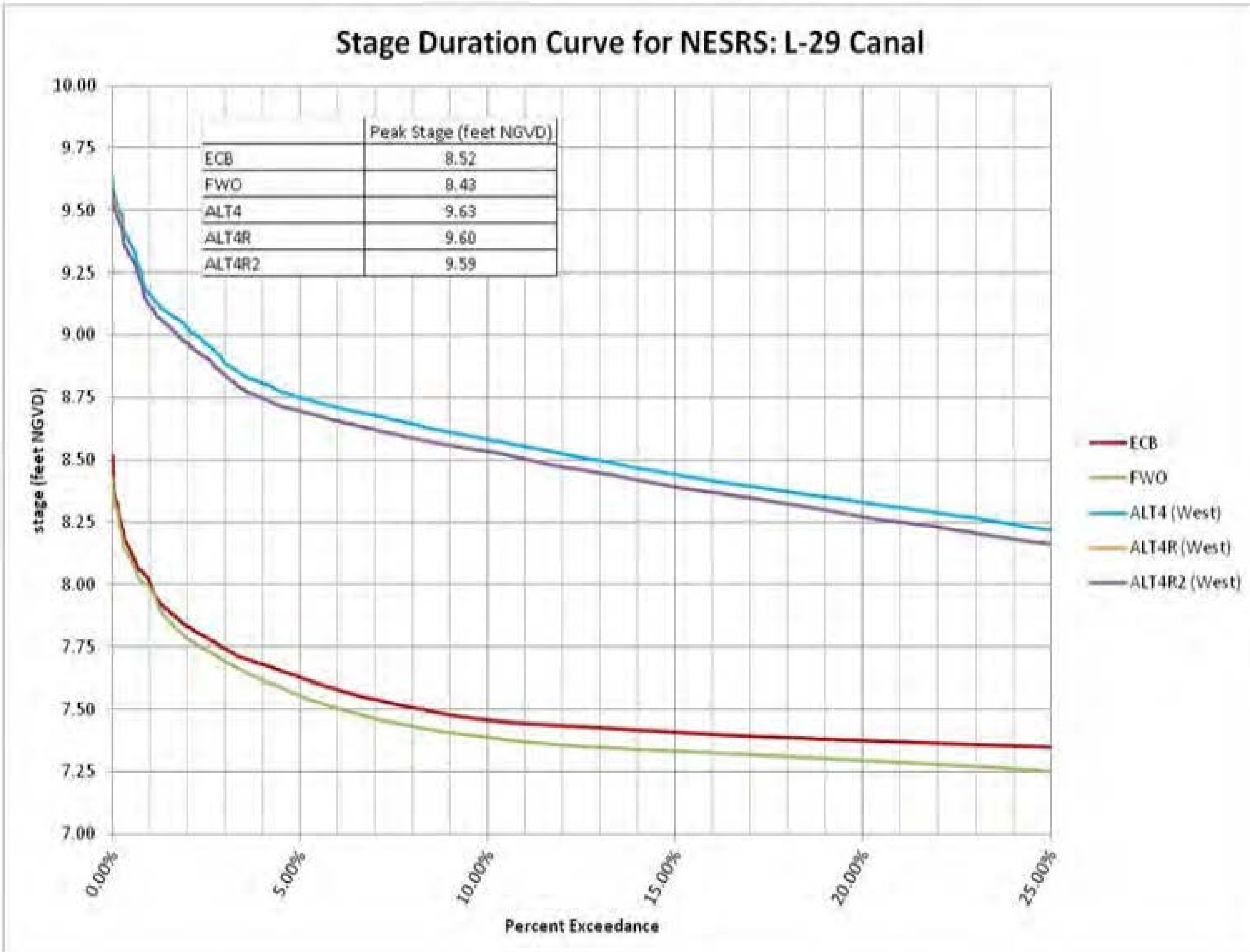


FIGURE A.8-22: L-29 CANAL STAGE DURATION CURVES FOR CEPP BASELINES AND CEPP ALTERNATIVES 4, 4R, AND 4R2 (UPPER 25%)

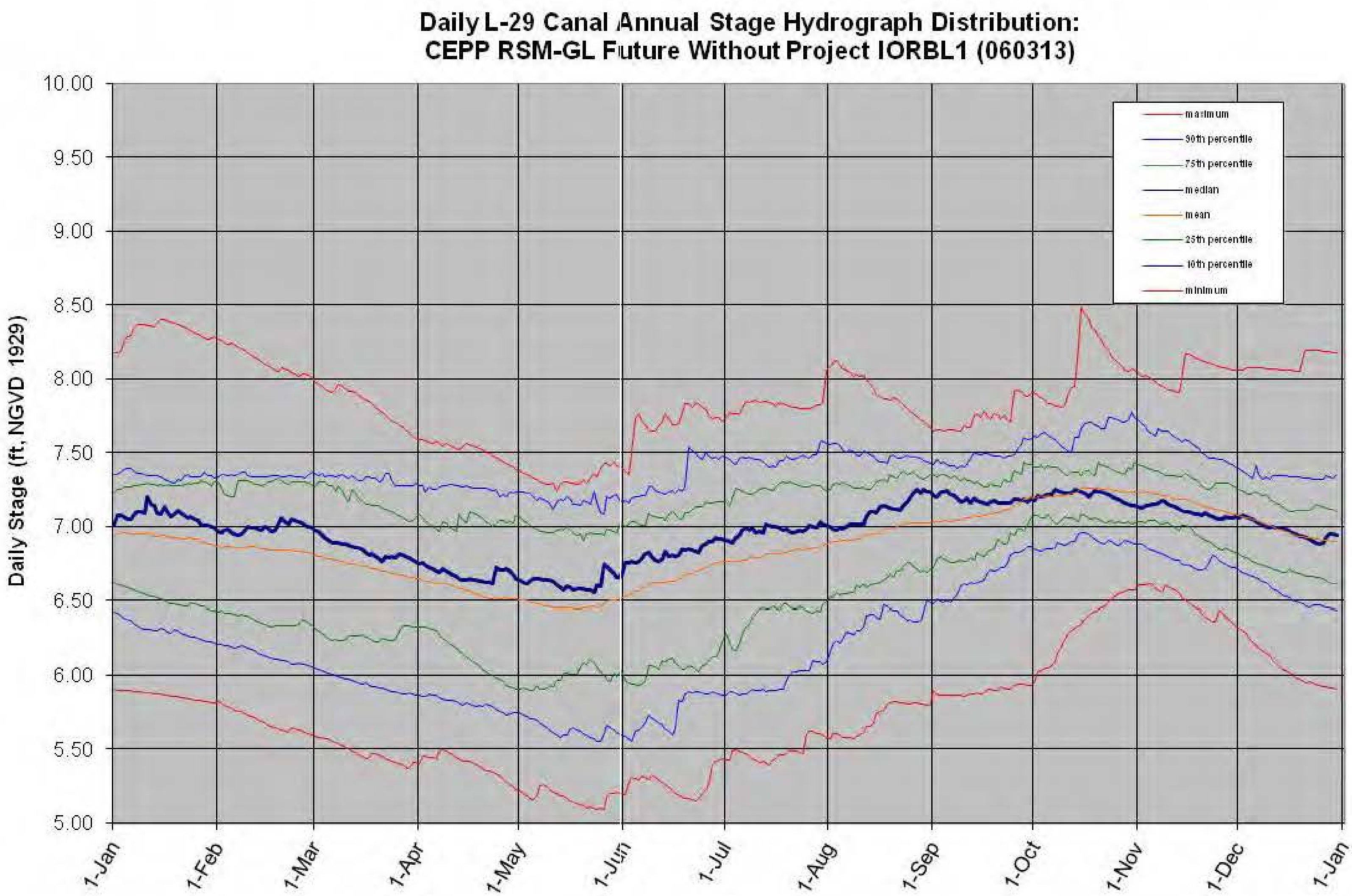


FIGURE A.8-23: L-29 CANAL ANNUAL AVERAGE STAGE HYDROGRAPHS FOR CEPP IORBL1 FUTURE WITHOUT PROJECT BASELINE

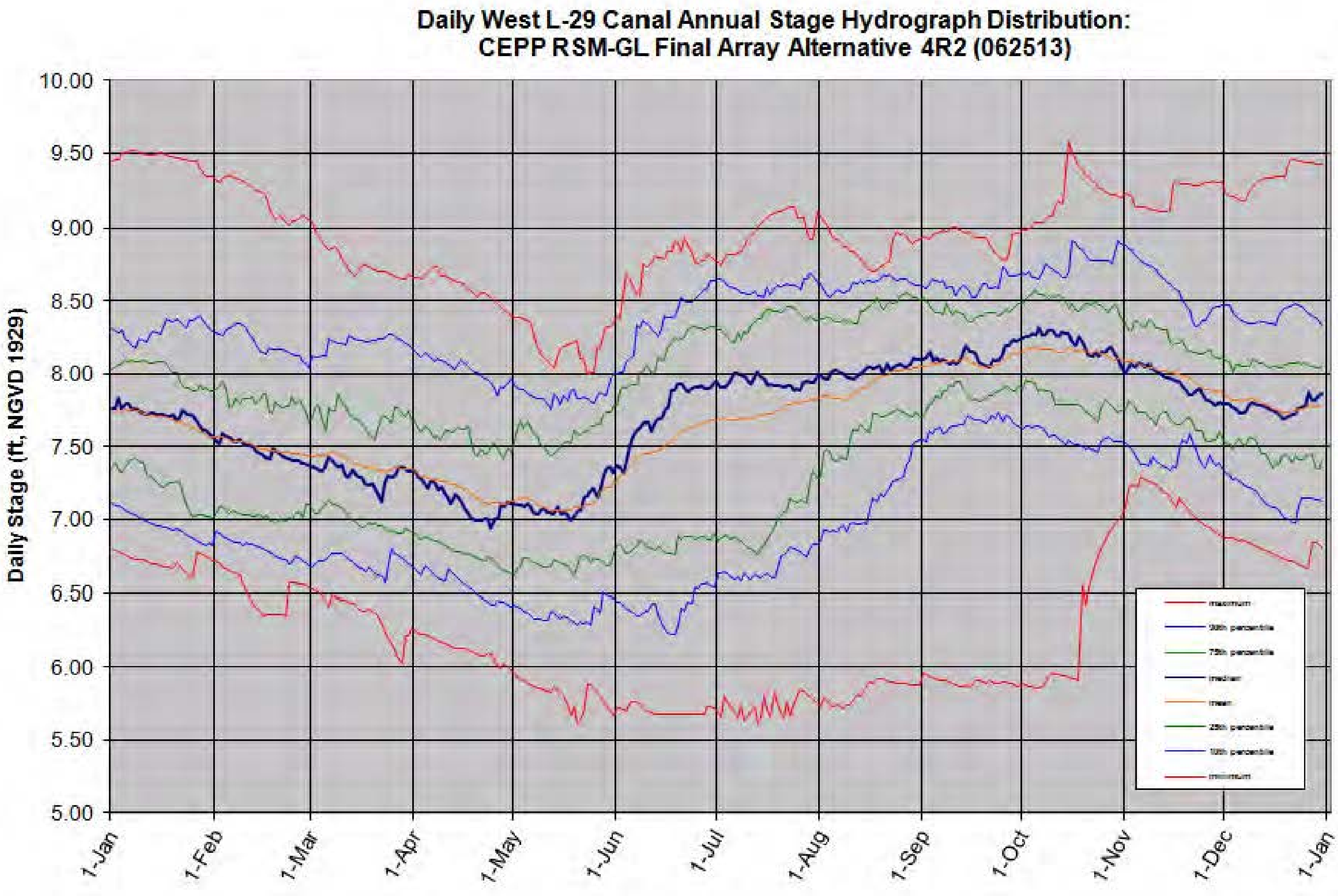


FIGURE A.8-24: WEST L-29 CANAL ANNUAL AVERAGE STAGE HYDROGRAPHS FOR CEPP ALTERNATIVE 4R2

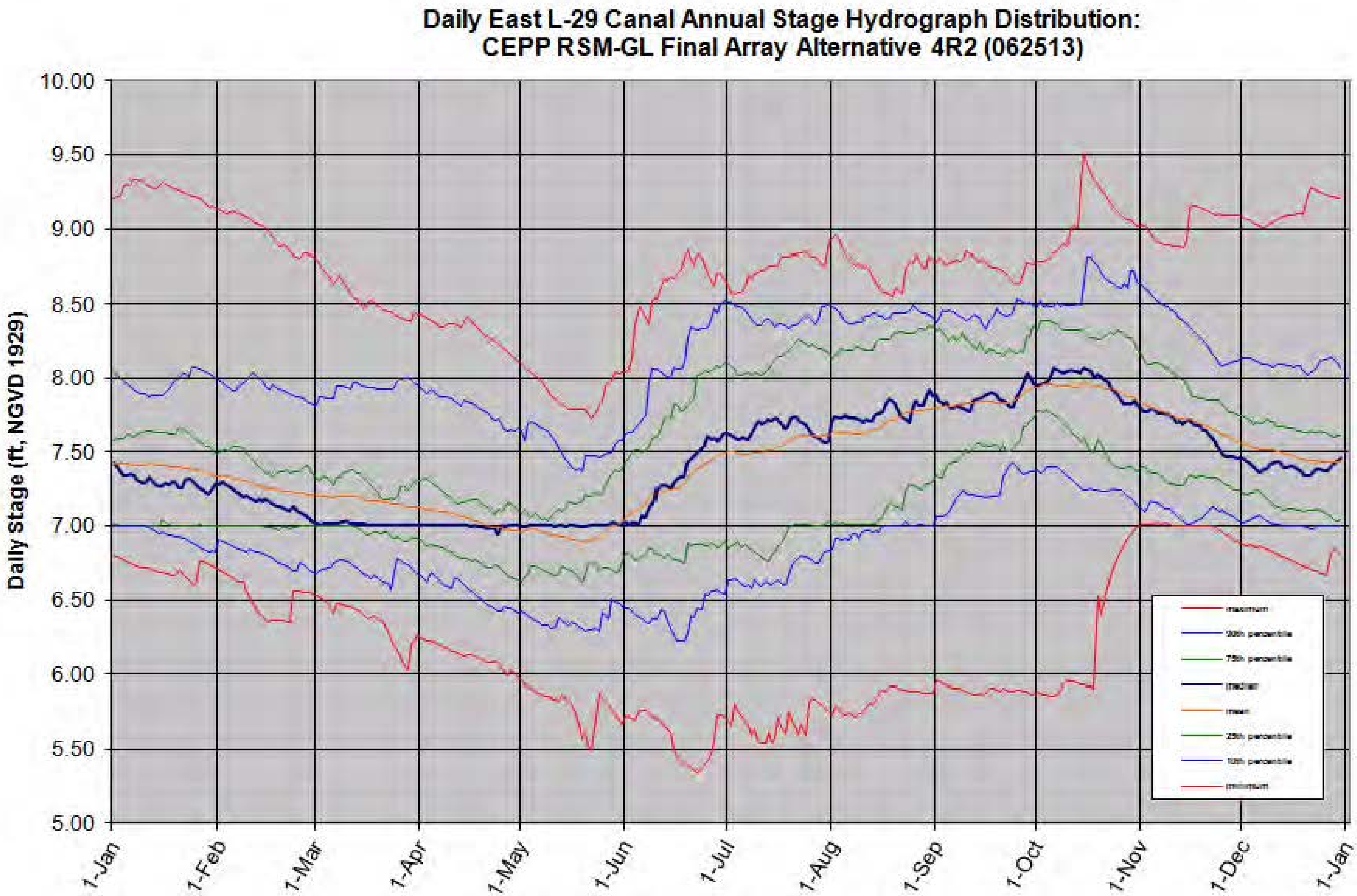


FIGURE A.8-25: EAST L-29 CANAL ANNUAL AVERAGE STAGE HYDROGRAPHS FOR CEPP ALTERNATIVE 4R2

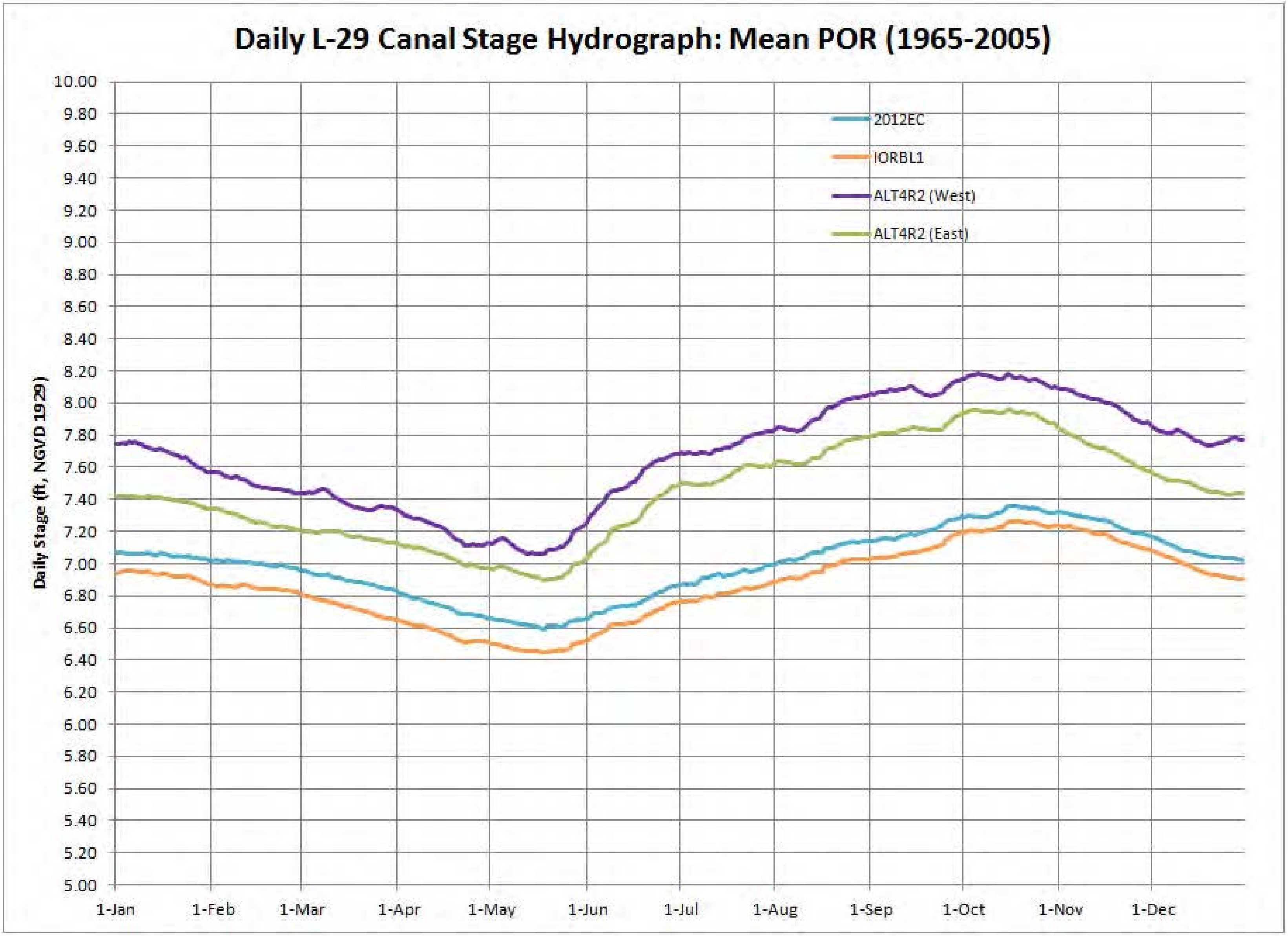


FIGURE A.8-26: L29 CANAL MEAN DAILY STAGE HYDROGRAPH FOR CEPP UPDATED BASELINES AND ALTERNATIVE 4R2

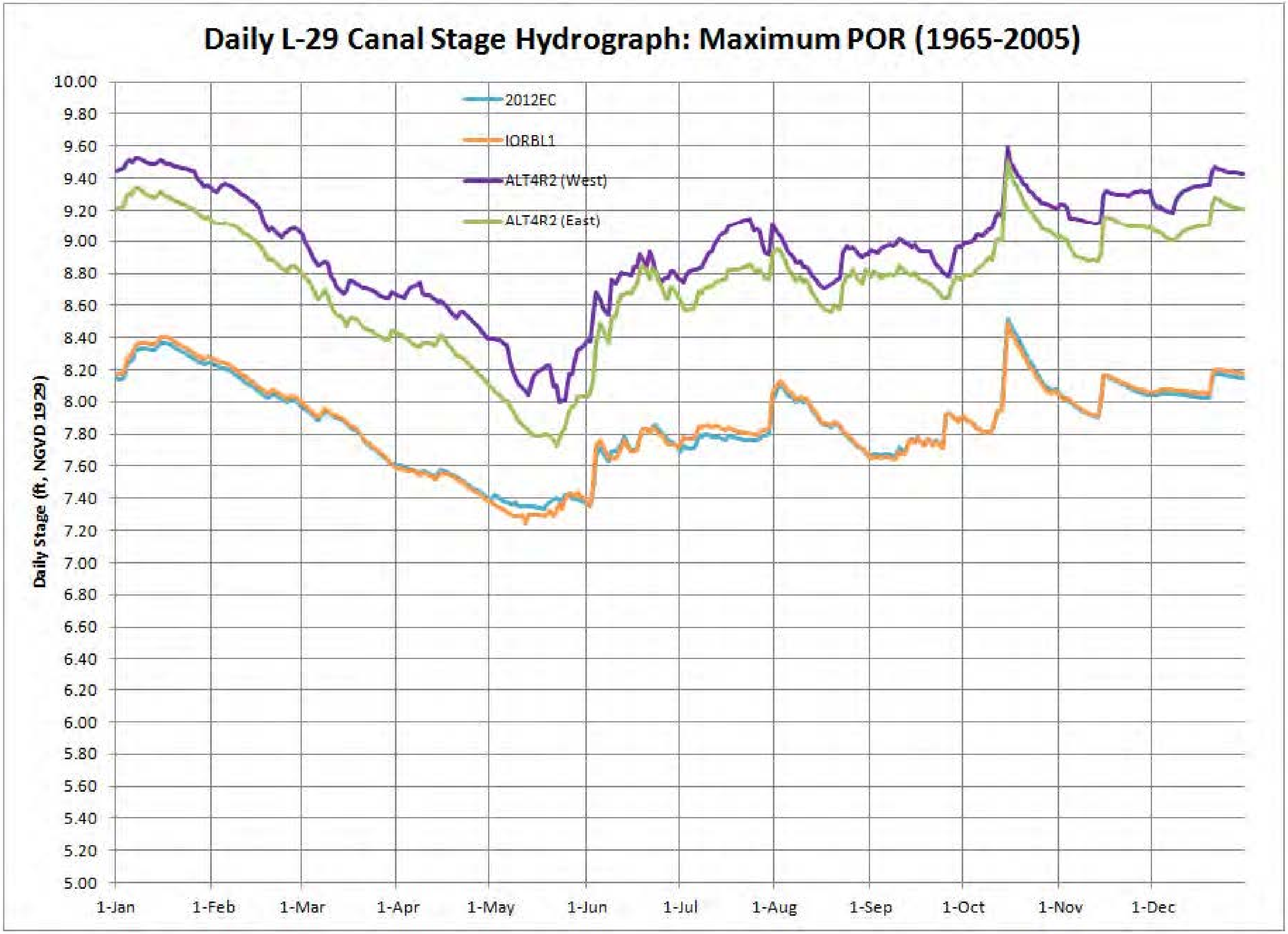


FIGURE A.8-27: L29 CANAL MAXIMUM DAILY STAGE HYDROGRAPH FOR CEPP UPDATED BASELINES AND ALTERNATIVE 4R2

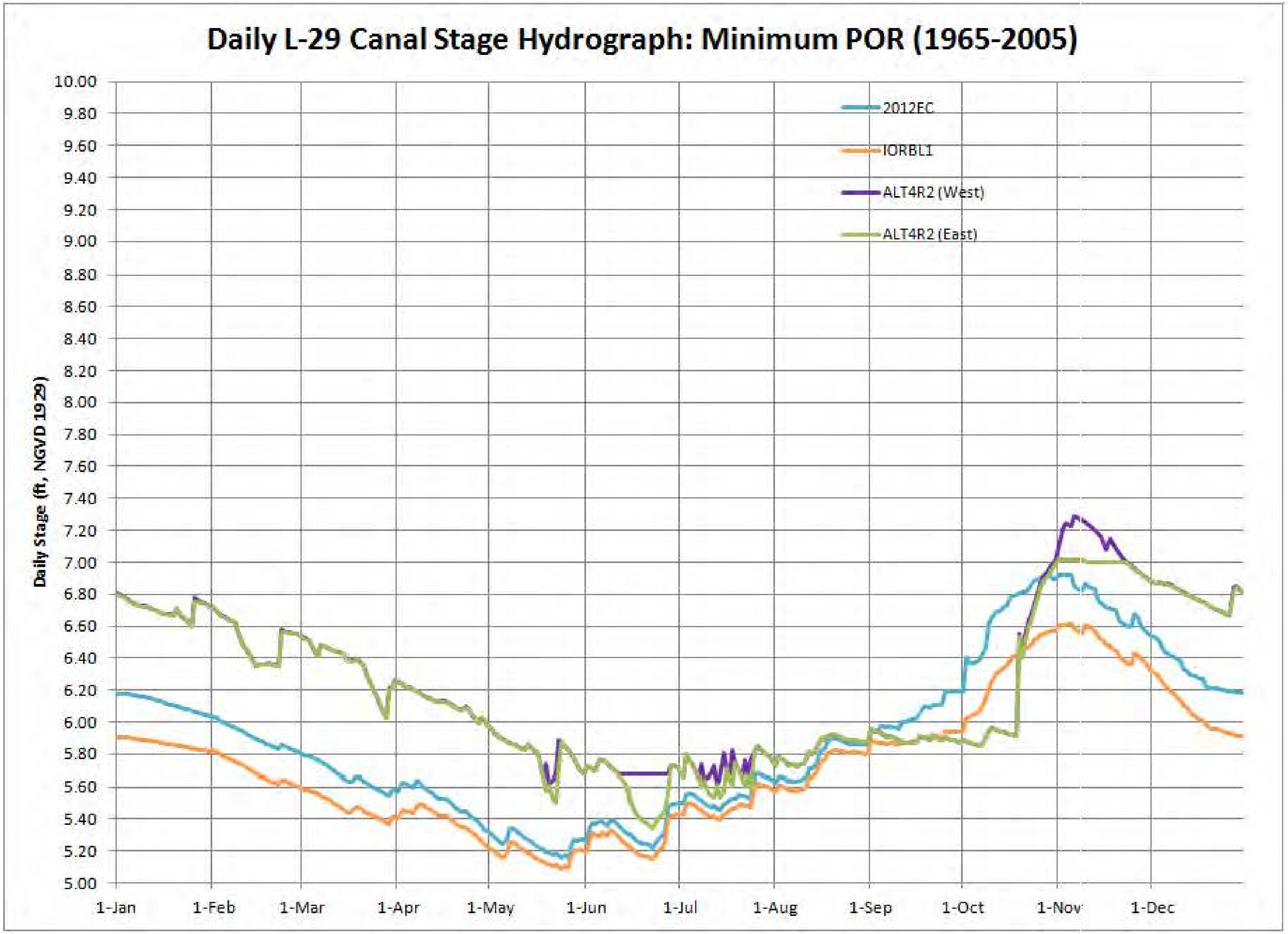


FIGURE A.8-28: L29 CANAL MINIMUM DAILY STAGE HYDROGRAPH FOR CEPP UPDATED BASELINES AND ALTERNATIVE 4R2

A.8.3.2.3 Lake Okeechobee Herbert Hoover Dike Design Considerations

A.8.3.2.3.1 Lake Okeechobee Assumptions for CEPP Future Without Project Condition

The CEPP existing condition and future without project condition assumption for the operation of Lake Okeechobee is 2008 Lake Okeechobee Regulation Schedule (2008 LORS); the complete existing condition and future without project condition assumptions for CEPP are documented in **Section 2** of the CEPP main PIR report. When it was approved in April 2008, the 2008 LORS was identified as an interim schedule. Independent of CEPP implementation, there is an expectation that revisions to the 2008 LORS will be needed following the implementation of other CERP projects and Herbert Hoover Dike infrastructure remediation. USACE expects to operate under the 2008 LORS until there is a need for revisions due to the earlier of either of the following actions: (1) system-wide operating plan updates to accommodate CERP "Band 1" Projects (described in Section 6.1.3.2 of the main PIR report) or (2) completion of sufficient HHD remediation for reaches 1, 2, and 3 and associated culvert improvements, as determined necessary to lower the DSAC rating from Level 1. The future Lake Okeechobee Regulation Schedule which may be developed in response to actions (1) and/or (2) is unknown at this time. At the start of the CEPP plan formulation process, the future without project condition adopted the 2008 LORS as a reasonable assumption since it would be speculation to change the operations plan based on future actions occurring independent of CEPP (e.g. HHD rehabilitation or CERP Band 1 project construction. The USACE had also determined during CEPP scoping that the expedited CEPP planning process and PIR would not be the mechanism to propose or conduct the required NEPA evaluation of modifications to the Lake Okeechobee Regulation Schedule. Until a new operating schedule is developed under a future study, the 2008 LORS is the best estimate for operations in the future without project condition.

Hydrologic modeling conducted for the CEPP future without project condition (FWO), which assumes no modifications to 2008 LORS and completion of the Kissimmee River Restoration, CERP C-43, and CERP C-44 restoration projects, indicated minor to moderate adverse effects due to increased frequency of low Lake Okeechobee stages and increased water supply cutbacks within the Lake Okeechobee Service Area (LOSA) and the Lower East Coast Service Areas (moderate improvements were also indicated for reduced frequency of high Lake Okeechobee stages).

The Herbert Hoover Dike (HHD) surrounds Lake Okeechobee, which is 720 square miles in size. The HHD was first authorized in 1930 and built by hydraulic dredge and fill methods. HHD has 143 miles of embankment with 5 spillway inlets, 5 spillway outlets, 32 Federal culverts, 9 navigation locks and 9 pump stations. There are structural integrity concerns with the embankment and internal culvert structures that resulted in a Dam Safety Action Classification (DSAC) risk rating of Level 1. DSAC Level 1 represents the highest USACE dam risk of failure rating and requires remedial action. The Major Rehabilitation Report (MRR) from 2000 divided the 143 mile dike into eight (8) Reaches with the initial focus on Reach 1. The current approved and planned remediation measures will address the highest points of potential failure in the system based on known areas of concern. These efforts are intended to lower the DSAC rating from Level 1. The CEPP future without project condition will assume the planned remediation of HHD will lower the DSAC risk rating and be completed by 2022. The following text provides the basis for this assumption.

Historically, the majority of embankment and foundation issues have occurred in Reaches 1, 2, and 3 related to one of the following primary potential failure modes: internal erosion through the embankment and internal erosion through the foundation. The additional failure modes associated with the culvert structures are: internal erosion along the conduits and internal erosion into the conduits.

Current approved HHD remediation measures consist of cutoff wall in Reach 1: cutoff wall task orders 1 through 9 are scheduled for completion in 2013, and 32 culvert replacements or removal around the lake are scheduled for completion in 2018. Planned remediation measures consist of cutoff wall and/or a seepage management system in Reaches 2 and 3. These actions are scheduled for completion in 2022. These remediation measures will not resolve all issues with the dam, nor will all current design criteria be met. To assess other issues and address future modifications with HHD, a comprehensive potential failure mode analysis and risk assessment is being performed and will be included in the ongoing Dam Safety Modification Report (DSMR). This report is scheduled for completion/approval in 2015.

Prior to the 2008 LORS, Lake Okeechobee operated under the Water Supply and Environmental Regulation Schedule (WSE). The 2006-2008 LORS study was initiated because of adverse environmental impacts that WSE had on the lake ecology. Dam safety was later added as a performance criterion since lowering of the lake, as the LORS study was pursuing, is one of the basic Interim Risk Reduction Measures implemented for deficient dams until appropriate remediation is effectuated. The WSE held Lake Okeechobee stages approximately 1.0 – 1.5 feet higher than the 2008 LORS under wet conditions. Studies for the remediation of HHD are based on the 2008 LORS, which was used as the basis for the development of the Standard Project Flood (SPF) condition. The SPF is the design condition used for the risk assessment and remediation to address internal erosion failure modes.

A.8.3.2.3.2 Lake Okeechobee Assumptions for CEPP Future With Project Condition

Lake Okeechobee is currently operated in accordance with the 2008 LORS and the 2008 Lake Okeechobee and EAA Water Control Plan. Independent of CEPP implementation, there is an expectation that revisions to the 2008 LORS will be needed following the implementation of other CERP projects and Herbert Hoover Dike infrastructure remediation. USACE expects to operate under the 2008 LORS until there is a need for revisions due to the earlier of either of the following actions: (1) system-wide operating plan updates to accommodate CERP “Band 1” Projects (described in Section 6.1.3.2 of the main PIR report) or (2) completion of sufficient HHD remediation for reaches 1, 2, and 3 and associated culvert improvements, as determined necessary to lower the DSAC rating from Level 1. The future Lake Okeechobee Regulation Schedule which may be developed in response to actions (1) and/or (2) is unknown at this time. In balancing the multiple project purposes, USACE will timely shift from the interim 2008 LORS to a new schedule with the intent to complete any necessary schedule modification or deviations concurrent with the completion of (1) or (2). The occurrences of both events (1) and (2) are assumed for the CEPP future with project condition and are expected to allow for greater operational flexibility of Lake Okeechobee, potentially including higher lake levels for increased water storage. CERP envisioned that changes to system operations may be required as groups of restoration components come on line and that updates to the system operating manual may be required at certain intervals of overall CERP implementation. The CEPP is composed of increments of project components that were identified in the CERP.

As a result of the CEPP preliminary screening process, the hydrologic modeling conducted for all CEPP alternatives (including the Recommended Plan Alternative 4R2) to optimize system-wide performance incorporated the current Regulation Schedule management bands of the 2008 LORS. The hydrologic modeling of the CEPP alternatives included proposed revisions to the 2008 LORS flow chart guidance of maximum allowable discharges, which are dependent on the following criteria:

- Class limits for Lake Okeechobee inflow and climate forecasts, including tributary hydrologic conditions, seasonal climate outlook, and multi-seasonal climate outlook;

- Stage level, as delineated by the Regulation Schedule management bands;
- Stage trends (whether water levels are receding or ascending).

Most of the 2008 LORS refinements applied in the CEPP modeling lie within the bounds of the operational limits and flexibility available in the current 2008 LORS, with the exception of the adjustments made to the class limits for the Lake Okeechobee inflow and climate forecasts. Under some hydrologic conditions, the class limit adjustments made to the Lake Okeechobee inflow and climate forecasts reduced the magnitude of allowable discharges from the Lake, thereby resulting in storage of additional water in the Lake in order to optimize system-wide performance and ensure compliance with Savings Clause requirements. However, these class limit changes represent a change in the flow chart guidance that extends beyond the inherent flexibility in the current 2008 LORS. Additional information and summary documentation of these assumptions can be found in Section A.8.3.2.3.3 and with the MDRs in Annex A-3 (LORS assumptions are described in Appendix B of each pertinent MDR).

CEPP benefits gained from sending new water south from Lake Okeechobee are derived in part from operational refinements that can take place within the existing, inherent flexibility of the 2008 LORS, and in part with refinements that are beyond the schedule's current flexibility. Modifications to 2008 LORS will be required to optimally utilize the added storage capacity of the A-2 FEB to send the full 210,000 ac-ft/yr of new water available in CEPP south to the Everglades, while maintaining compliance with Savings Clause requirements for water supply and flood control performance levels.

It is anticipated that the need for modifications to the 2008 LORS will be initially triggered by non-CEPP actions and that these actions will occur earlier than implementation of CEPP. Therefore, the CEPP PIR, including the Project Operating Manual (POM), will not be the mechanism to propose or conduct the required NEPA evaluation of modifications to the Lake Okeechobee Regulation Schedule. However, depending on the ultimate outcome of these future Lake Okeechobee Regulation Schedule revisions, including the level of inherent operational flexibility provided with these revisions, CEPP implementation may still require further Lake Okeechobee Regulation Schedule revisions to optimize system-wide performance and ensure compliance with Savings Clause requirements.

Consistent with this rationale for Lake Okeechobee operational modifications within the CEPP future with project alternatives, ecological performance measures for Lake Okeechobee were not included as part of the habitat unit ecological benefits evaluation for CEPP (peer reviewed and approved by the ECO-PCX). The environmental effects to Lake Okeechobee are assumed to be approximately equivalent across the CEPP future with project alternatives, and the simulated hydrologic and ecological effects to Lake Okeechobee are included as part of the environmental effects documentation in the CEPP PIR.

A.8.3.2.3.3 Lake Okeechobee Modeling Assumptions and Performance Results

Lake Okeechobee stage duration curves for the RSM-BN model representation of the CEPP ECB (LORS 2008), CEPP FWO (LORS 2008, plus additional CERP and non-CERP projects), and CEPP alternatives 1 through 4R2 (LORS 2008, additional CERP and non-CERP projects, and prescribed assumed operational flexibility) are included as Figure A.8-29 and Figure A.8-30. A single RSM-BN simulation was originally completed for all of the CEPP components north of the red line for the final array of alternatives 1 through 4 However, during the modeling effort for the Alt 4R and Alt 4R2, revised RSM-BN simulations were completed for these alternative simulations to address performance shortfalls observed with Alt 4 and Alt 4M, including to avoid potential impacts to water supply levels of service in the LOSA and the

LEC and to avoid increases in the number of low flow events to the St. Lucie Estuary. The revised RSM-BN simulations resulted in updated boundary conditions for the RSM-GL modeling of Alt 4R and Alt 4R2. A summary of the Alt 4R2 modeling assumptions for the Lake Okeechobee Regulation Schedule releases, compared to the 2008 LORS, are indicated on Figure A.8-31 and Tables A.8-2 through A.8-5 (note: breakpoints for the 2008 LORS Regulation Schedule zones/bands were not otherwise modified for Alt 4R2 and remain consistent with Figure A.8.32); complete documentation is provided in the Hydrologic Modeling Annex A-2. Peak stages for the CEPP baselines and the CEPP final array of alternatives are summarized as follows: 17.54 feet NGVD for the ECB; 17.50 feet NGVD for the FWO; 17.64 for the CEPP alternatives 1 through 4, 4R, and 4R2; and 17.66 for CEPP Recommended Plan Alternative 4R2. The CEPP baselines and the CEPP alternatives all show simulated stages above 17.25 feet NGVD: 18 days for the ECB; 9 days for the FWO; 23 days for the CEPP alternatives 1 through 4; and 29 days for Alternative 4R and Alternative 4R2 (note: 14,975 days in the RSM-BN 41-year period of simulation). The LORS 2008 EIS assessment recognized that minimizing the frequency of exceedance of the 17.25 feet elevation offers additional protection for public safety and the HHD, for the condition prior to completion of the current approved and planned HHD remediation measures, and this criterion was evaluated as a LORS project performance measure. Significant increases in the frequency, duration, and magnitude of Lake Okeechobee peak stages do not result from the assumed operational flexibility with the CEPP alternatives (including Alternative 4R2), despite the assumed completion of HHD remediation measures, because the adverse ecological effects associated with increased lake stages and the associated increases in high volume releases to the estuaries were effectively balanced during the CEPP preliminary screening (for additional discussion of screening metrics, refer to **Section 3** of the PIR main report). Following completion of the HHD remediation of Reaches 1, 2, and 3, the degree to which higher maximum lake stages and increased frequency and duration of high lake stages would be considered as potentially viable by the USACE, if at all, will be contingent on the conclusions identified in the 2015 DSMR (note: this process is independent and separate from the CEPP project).

Given recognition of the DSMR uncertainty and the continued utilization of the LORS 2008 Lake Okeechobee Regulation Schedule for CEPP, EN-W assessment of the Lake Okeechobee high water performance with CEPP (Alternatives 1 through 4R2) indicated consistency with the HHD formulation assumptions established for the CEPP FWO condition, which included general consideration of potential risk and uncertainty associated with increased lake stages. Lake Okeechobee high water performance requirement will likely need to be revisited following completion of the 2015 DSMR, but the CEPP stage duration curve trends for increased high water conditions appear reasonable based on the USACE current best available information and current expectations for the HHD remediation.

Extreme high lake stages have also been documented to adversely impact the plant and animal communities, through processes which include the following: physical uprooting of emergent and submerged plants; reduced light levels in the water column due to increased suspended sediment; and littoral zone exposure to increased nutrient levels from the water column. The frequency of occurrence for lake stages above 16.0 feet, 16.5 feet, 17.0 feet, and 17.25 feet are summarized in Figure A.8-33. Lake Okeechobee stages between 16.0 and 17.25 feet NGVD correspond to the seasonal range of the top zone of the 2008 LORS Regulation Schedule, and this performance metric was considered by the USACE during the LORS Regulation Schedule study.

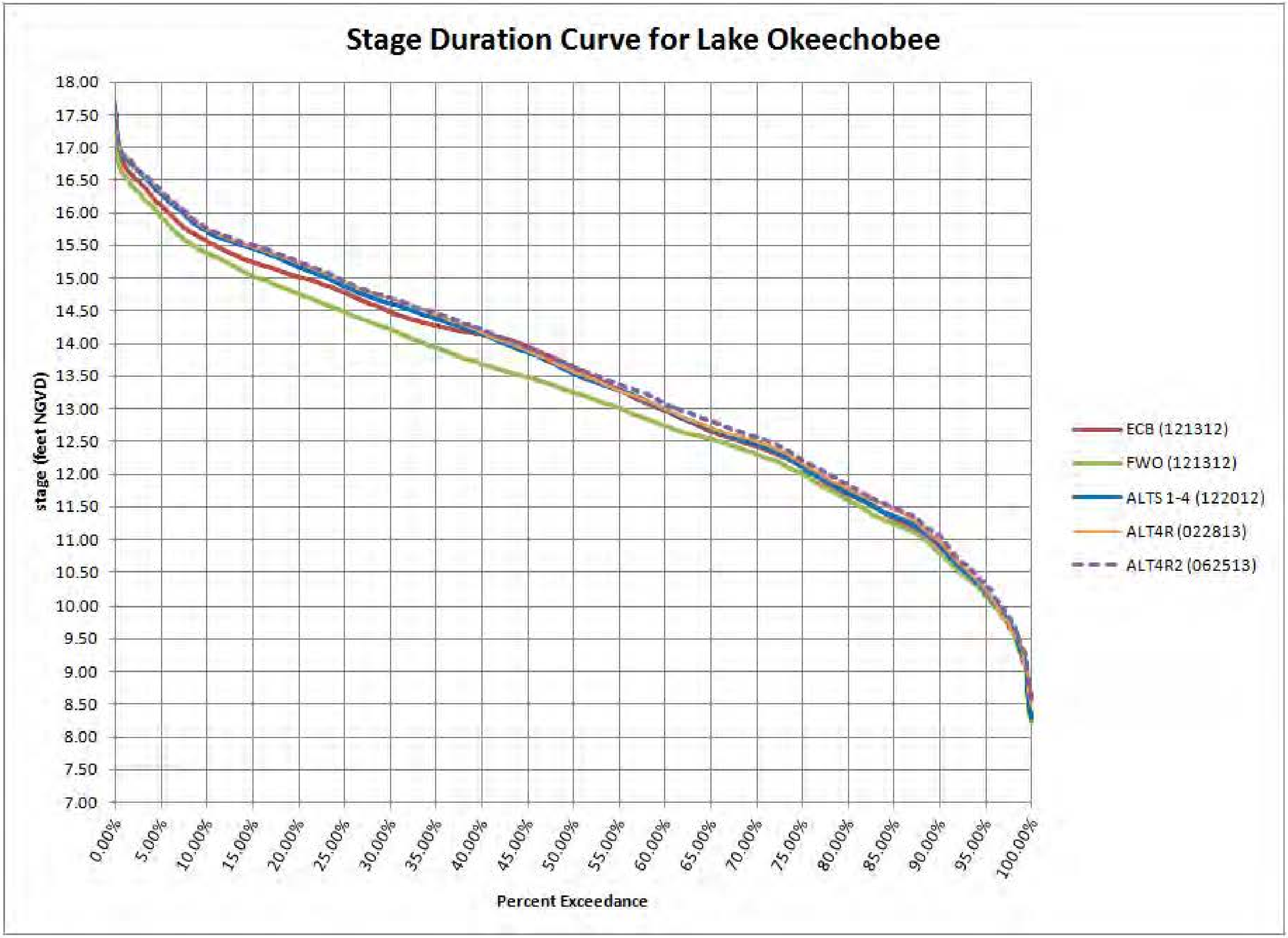


FIGURE A.8-29: LAKE OKEECHOBEE STAGE DURATION CURVES FOR CEPP BASELINES AND CEPP ALTERNATIVES 1 THROUGH 4R2

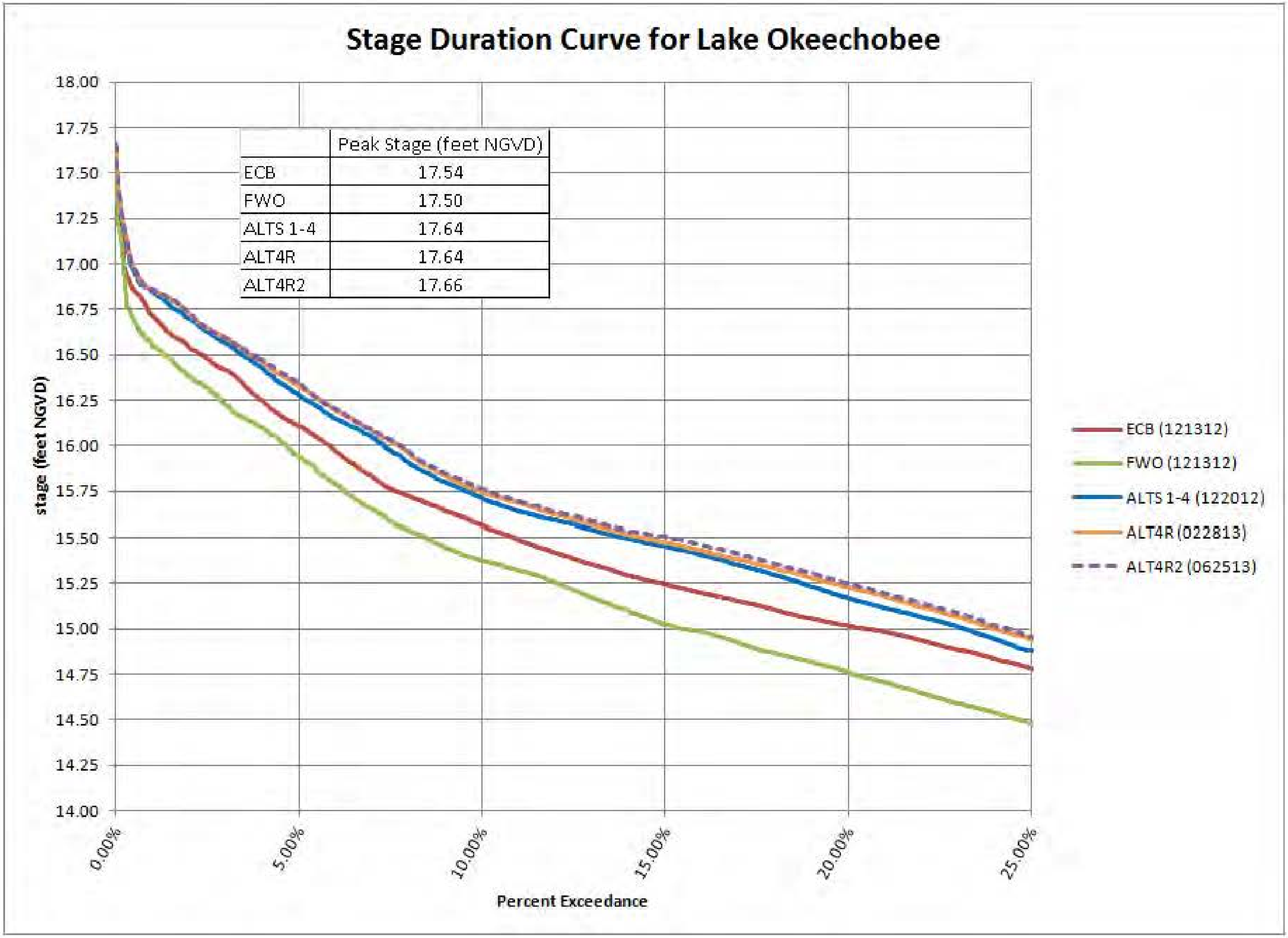


FIGURE A.8-30: LAKE OKEECHOBEE STAGE DURATION CURVES FOR CEPP BASELINES AND CEPP ALTERNATIVES 1 THROUGH 4R2 (UPPER 25%)

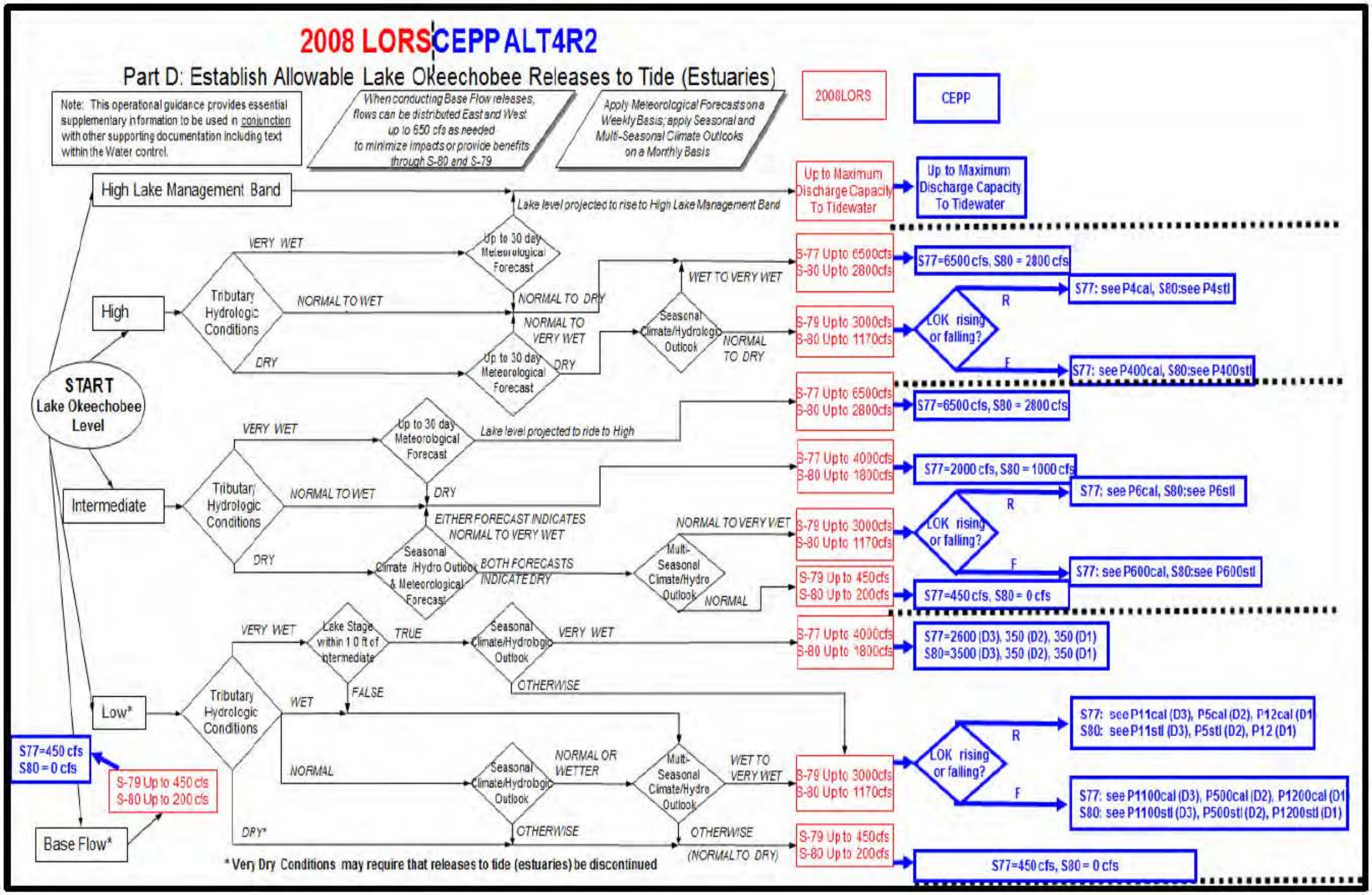


FIGURE A.8-31: LAKE OKEECHOBEE REGULATION SCHEDULE RELEASE ASSUMPTIONS FOR ALT 4R2

TABLE A.8-2: LAKE OKEECHOBEE REGULATION SCHEDULE CLASSIFICATION ASSUMPTIONS FOR ALT 4R2 TRIBUTARY HYDROLOGIC CONDITIONS

LORS2008

Classification of Lake Okeechobee Tributary Hydrologic Conditions

Palmer Index Class Limits	2-wk Mean LO Inflow Class Limit	Tributary Hydrologic Classification*
> 3.0	>= 6000 cfs	Very Wet
1.5 to 2.99	2500 - 5999 cfs	Wet
-1.49 to 1.49	500 - 2499 cfs	Near Normal
-2.99 to -1.5	-5000 - 500 cfs	Dry
-3.0 or less	< -5000 cfs	Very Dry

*use the wettest of the two indicators

CEPP-RSMBN

Classification of Lake Okeechobee Tributary Hydrologic Conditions

Palmer Index Class Limits	2-wk Mean LO Inflow Class Limit	Tributary Hydrologic Classification*
> 3.3	>= 8700 cfs	Very Wet
0.01 to 3.29	1000 - 8699 cfs	Wet
-0.49 to 0.0	500 - 999 cfs	Near Normal
-2.99 to -0.50	-5000 - 500 cfs	Dry
-3.0 or less	< -5000 cfs	Very Dry

*use the wettest of the two indicators

TABLE A.8-3: LAKE OKEECHOBEE REGULATION SCHEDULE CLASSIFICATION ASSUMPTIONS FOR ALT 4R2 NET INFLOW SEASONAL OUTLOOK

LORS2008

Classification of Lake Okeechobee Net Inflow Seasonal Outlook**

Lake Net Inflow Prediction (million acre-feet) (Does not include ET)	Equivalent Depth (feet)	Lake Okeechobee Net Inflow Seasonal Outlook
> 0.93	> 2.0	Very Wet
0.71 to 0.93	1.51 to 2.0	Wet
0.35 to 0.70	0.75 to 1.5	Normal
< 0.35	< 0.75	Dry

**volume-depth conversion based on average lake surface area of 467,000 acres.

CEPP-RSMBN

Classification of Lake Okeechobee Net Inflow Seasonal Outlook**

Lake Net Inflow Prediction (million acre-feet) (Does include ET)	Equivalent Depth (feet)	Lake Okeechobee Net Inflow Seasonal Outlook
> 1.43	> 3.06	Very Wet
0.91 to 1.43	1.93 to 3.06	Wet
0.46 to 0.90	0.98 to 1.92	Normal
< 0.46	< 0.98	Dry

**volume-depth conversion based on average lake surface area of 467,000 acres.

TABLE A.8-4: LAKE OKEECHOBEE REGULATION SCHEDULE CLASSIFICATION ASSUMPTIONS FOR ALT 4R2 NET INFLOW MULTI-SEASONAL OUTLOOK

LORS2008

Classification of Lake Okeechobee Net Inflow Multi-Seasonal Outlook**

Lake Net Inflow Prediction (million acre-feet) (Does not include ET)	Equivalent Depth (feet)	Lake Okeechobee Net Inflow Multi-Seasonal Outlook
> 2.0	> 4.3	Very Wet
1.18 to 2.0	2.51 to 4.3	Wet
0.5 to 1.17	1.1 to 2.5	Normal
< 0.5	< 1.1	Dry

**volume-depth conversion based on average lake surface area of 467,000 acres.

CEPP-RSMBN

Classification of Lake Okeechobee Net Inflow Multi-Seasonal Outlook**

Lake Net Inflow Prediction (million acre-feet) (Does include ET)	Equivalent Depth (feet)	Lake Okeechobee Net Inflow Multi-Seasonal Outlook
> 2.0	> 4.3	Very Wet
1.18 to 2.0	2.51 to 4.3	Wet
0.6 to 1.17	1.3 to 2.5	Normal
< 0.6	< 1.3	Dry

**volume-depth conversion based on average lake surface area of 467,000 acres.

TABLE A.8-5: LAKE OKEECHOBEE REGULATION SCHEDULE CLASSIFICATION ASSUMPTIONS FOR ALT 4R2 PULSE RELEASES TO CALOOSAHATCHEE ESTUARY AND SAINT LUCIE ESTUARY

CEPP-RSMBN ALT4R2:-Pulse releases (as a function of Lake level) from Lake Okeechobee into Caloosahatchee estuary in cubic feet per second (cfs).

	High		Intermediate		Low					
zone	A	A	B	B	D3	D3	D2	D2	D1	D1
Day of Pulse	R=Rising P4cal	F=Falling P400cal	R=Rising P6cal	F=Falling P600cal	R=Rising P11cal	F=Falling P1100cal	R=Rising P5cal	F=Falling P500cal	R=Rising P12cal	F=Falling P1200cal
1	500	125	840	504	125	125	850	850	504	504
2	1,375	343.75	1,920	1,152	344	344	2,200	2,200	1,152	1,152
3	1,625	406.25	2,220	1,332	406	406	2,600	2,600	1,332	1,332
4	1,250	312.50	1,680	1,008	313.50	313.50	2,000	2,000	1,008	1,008
5	1,000	250	1,440	864	250	250	1,600	1,600	864	864
6	750	187.50	1,140	684	188.50	188.50	1,200	1,200	684	684
7	500	125	960	576	125	125	850	850	576	576
8	250	62.50	720	432	62.50	62.50	500	500	432	432
9	125	31.25	540	324	31.25	31.25	350	350	324	324
10	125	31.25	540	324	31.25	31.25	350	350	324	324
Average Flow (cfs)	750	187.5	1200	720	187.7	187.7	1250	1250	720	720
Volume (ac-ft)	14,873	3,718	23,796	14,278	3,722	3,722	24,788	24,788	14,278	14,278
†equivalent depth (ft)	0.03	0.01	0.05	0.03	0.01	0.01	0.05	0.05	0.03	0.03

† : Volume to depth Conversion based on average Lake surface area of 467,000 acres.

CEPP-RSMBN ALT4R2:-Pulse releases (as a function of Lake level) from Lake Okeechobee into St Lucie estuary in cubic feet per second (cfs).

	High		Intermediate		Low					
zone	A	A	B	B	D3	D3	D2	D2	D1	D1
Day of Pulse	R=Rising P4stl	F=Falling P400stl	R=Rising P6stl	F=Falling P600stl	R=Rising P11stl	F=Falling P1100stl	R=Rising P5stl	F=Falling P500stl	R=Rising P12stl	F=Falling P1200stl
1	450	112.50	720	432	112.50	112.50	750	750	432	432
2	600	150	960	576	150	150	1,000	1,000	576	576
3	525	131.25	840	504	131.25	131.25	900	900	504	504
4	375	93.75	600	360	93.75	93.75	600	600	360	360
5	250	62.50	420	252	62.50	62.50	450	450	252	252
6	225	56.25	360	216	56.25	56.25	350	350	216	216
7	150	37.50	240	144	37.50	37.50	250	250	144	144
8	150	37.50	240	144	37.50	37.50	250	250	144	144
9	100	25	0	0	25	25	200	200	0	0
10	100	25	0	0	25	25	0	0	0	0
Average Flow (cfs)	292.5	73.1	438.0	262.8	73.1	73.1	475.0	475.0	262.8	262.8
Volume (ac-ft)	5,800	1,450	8,686	5,211	1,450	1,450	9,419	9,419	5,211	5,211
†equivalent depth (ft)	0.01	0.00	0.02	0.01	0.00	0.00	0.02	0.02	0.01	0.01

† : Volume to depth Conversion based on average Lake surface area of 467,000 acres.

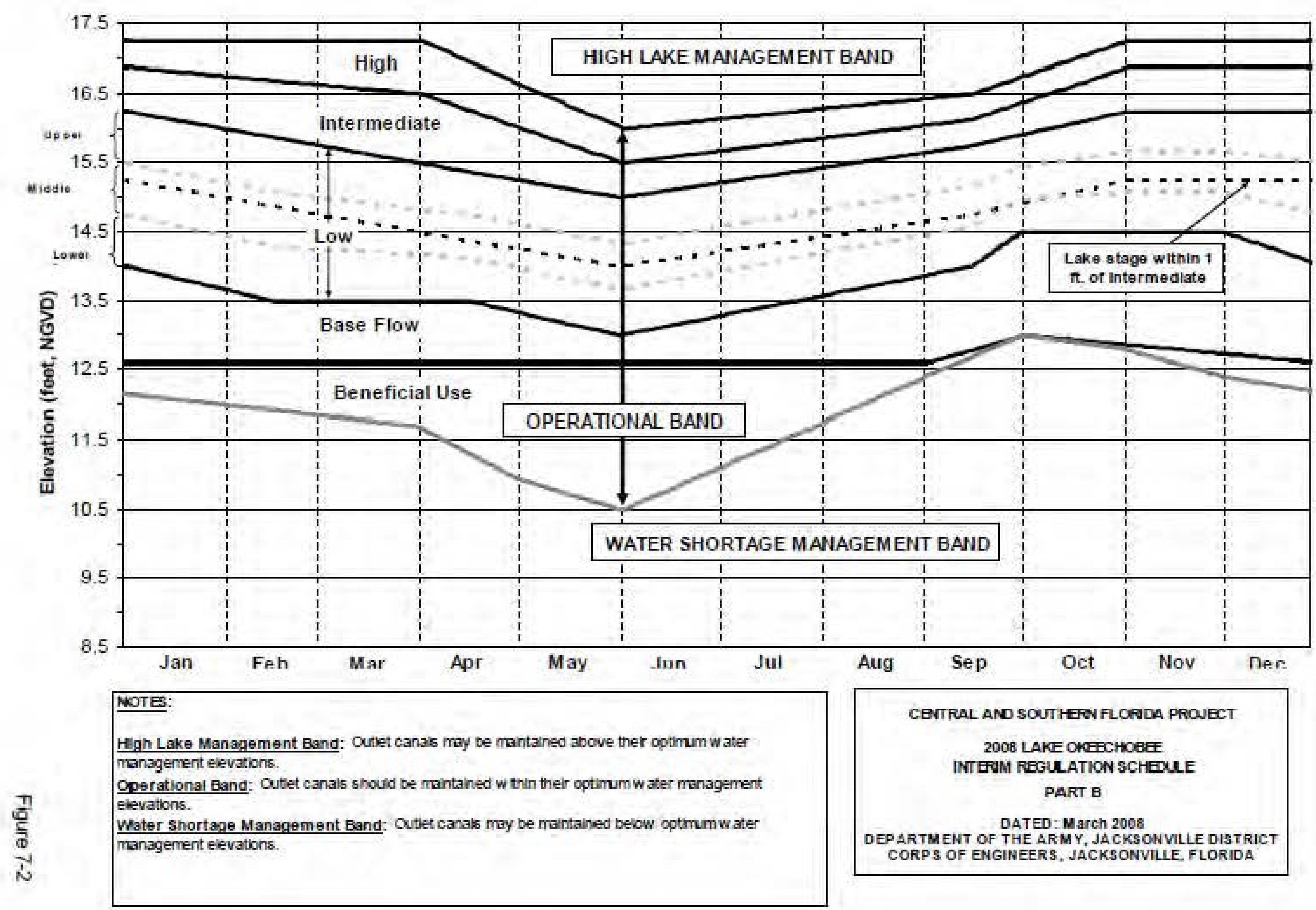


FIGURE A.8-32: LORS 2008 LAKE OKEECHOBEE REGULATION SCHEDULE

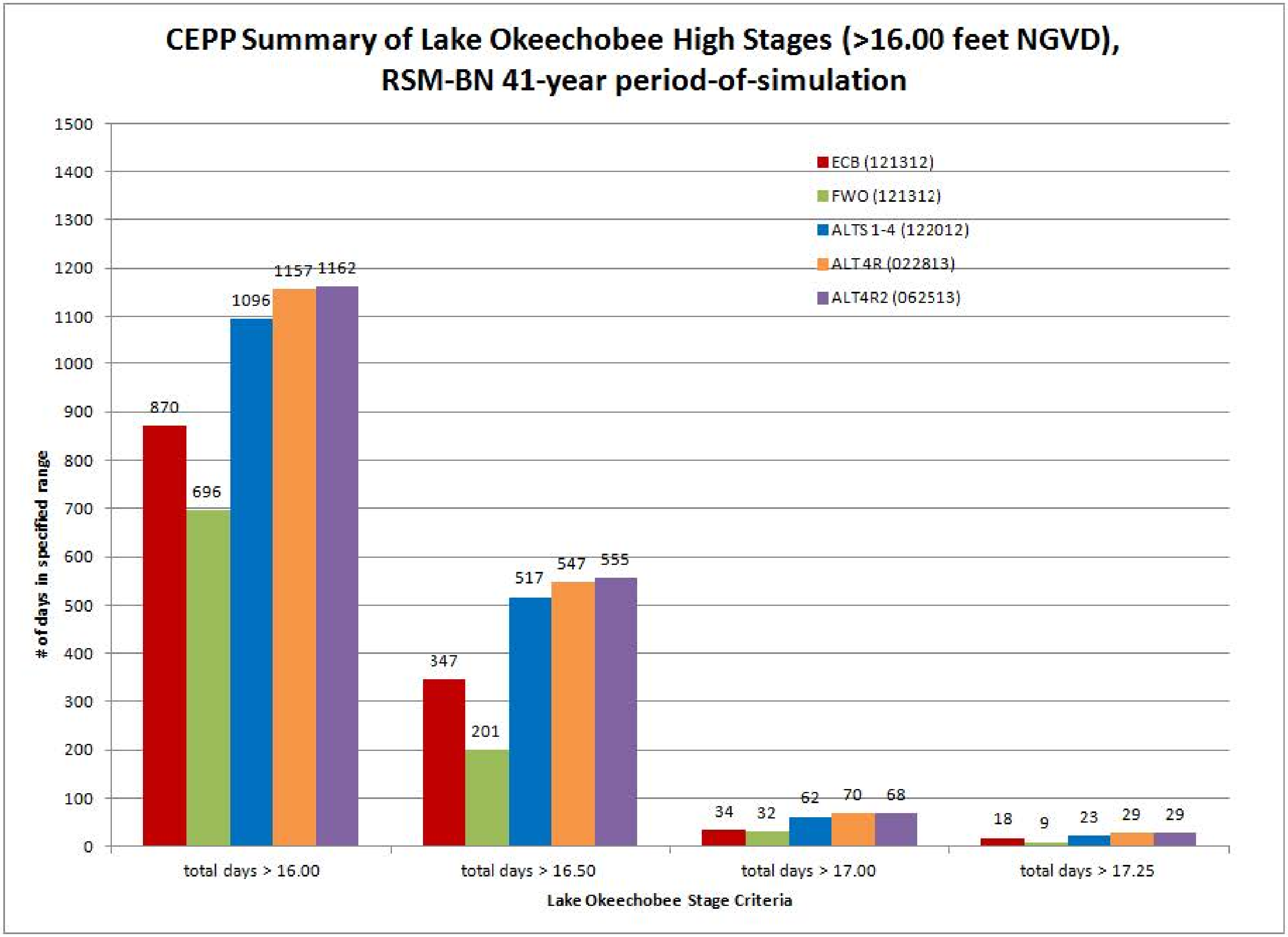


FIGURE A.8-33: OCCURRENCE FREQUENCY OF LAKE OKEECHOBEE HIGH STAGES

A.8.3.2.4 Flow Equalization Basin Design Considerations

Consistent with CEPP modeling assumptions for the action alternatives, operational stages for the EAA FEB storage feature were typically managed between 1 and 3 feet depth, with no additional structural inflows from Lake Okeechobee allowed when the FEB depth exceeded 3.8 feet. Structural inflows to the FEB would be discontinued when depths exceed 4 feet, although additional rainfall may further increase stages. Hydraulic design of the FEB perimeter levee system included consideration of the stage variability for FEB operations. Within the RSM-BN modeling conducted to support the CEPP preliminary screening and alternative evaluations, the SFWMD Restoration Strategies FEB located on the EAA A-1 parcel and the CEPP FEB on the EAA A-2 parcel are represented as a single storage feature. The purpose of the A-2 FEB, which will be operated in conjunction with the use of the A-1 FEB is to capture additional water from Lake Okeechobee for delivery to the Everglades, while maintaining the pre-project capability to provide water quality treatment for the existing EAA runoff and limited Lake Okeechobee discharges. The integrated FEB operations will be able to accept and provide some limited water quality pre-treatment of additional water from Lake Okeechobee during off-peak times, such as the dry season, when treatment capacity is available in the downstream STAs.

Stage duration curves for the CEPP FEB are shown in Figure A.8-34 for the updated FWO (IORBL1; 14k acre A-1 FEB only), and the CEPP final array alternatives (28k acre combined A-1 and A-2 FEB): Alternatives 1-4, Alternative 4R, and Alternative 4R2. The FWO results for stage in the FEB are not displayed since the RSM-BN modeling was not yet consistent with the project intent; additional details regarding the assumptions for the IORBL1 are provided in Section A.8.2.5. Ground surface elevations within the FEB were assumed at 9.63 feet NGVD for the RSM-BN modeling (note: the CEPP hydraulic design of the A-2 FEB assumed a different natural marsh grade of 9.0 feet NGVD for the A-2 FEB; however, RSM-BN model-simulated stages within the FEB were not utilized during the CEPP hydraulic design).

A.8.3.2.5 Quantification of Redline Flow Volumes and Timing

The recommended plan will provide approximately 214,000 ac-ft per year of additional water flow (based on comparison of Alternative 4R2 against the IORBL1) to the Everglades by redirecting through the EAA water which is currently being discharged to tide via the St. Lucie and Caloosahatchee Estuaries and providing FEB storage to attenuate flow rates, prior to water quality treatment using available, off-peak capacity of the state-operated STA-2 and STA-3/4; note that a flow increase of 210,000 ac-ft was identified during CEPP formulation, based on comparison of Alternative 4R2 against the FWO. Following water quality treatment, this additional flow quantity will be re-distributed as inflows to WCA 2A and WCA 3A, and the recommended plan features will modify the quantity, quality, timing, and spatial distribution of flows into and through WCA 3A, WCA 3B, and ENP to Florida Bay in order to meet the project objectives. This plan would be accomplished by a combination of modifications to the existing Central and South Florida project components, construction of additional components, and modifications to current approved water control manuals. Several proposed or existing levees, canals, and culverts, and pump stations would be constructed, modified, or removed to improve the flow of water through the system as the first increment of CEPP.

Features in the EAA (North of the Redline) include construction of the 14,000 acre A-2 FEB (L-624 perimeter levee and L-625 interior levee; C-624, C-624E, C-626 internal distribution channels; S-623, S-624, S-628 inlet structures; S-625 outlet structures, and C-625E, C-625W canals and channels connecting the FEB to the Miami Canal). Operation of the A-2 FEB would be integrated with the operation of the A-1 FEB, a state-funded and state-constructed FEB.

Conveyance features in WCA 2A and northern WCA 3A (South of the Redline) include: S-620, a gated culvert to deliver water from the L-6 Canal to the remnant L-5 Canal; S-622, a new gated spillway to deliver water from the remnant L-5 canal to the western L-5 canal (during L-6 diversion operations); S-621, a new gated spillway to deliver water from STA 3/4 to the S-7 pump station during peak discharge events (eastern flow route is not typically used during normal operations, including L-6 diversion operations; conveyance improvements to approximately 13.6 miles of the L-5 Canal; degrade approximately 2.9 miles of the southern L-4 Levee along the northwest boundary of WCA-3A; S-630, a 360 cfs pump station to maintain water supply deliveries west of the L-4 Canal to the Seminole Tribe of Florida's Big Cypress Reservation and STA-5/6; S-8A new gated culverts to deliver water from the Miami Canal (downstream of S-8, which pulls water from the L-5 Canal) to the L-4 Canal; and backfill approximately 13.5 miles of the Miami Canal and include upland mounds, between a point approximately 1.5 miles south of the S-8 pump station and Interstate Highway I-75.

The base condition assumptions for the CEPP ECB and FWO were established during the preliminary screening process from approximately December 2011 through February 2012, with the first preliminary base condition simulations with RSM-BN and RSM-GL released in May 2012. In order to maintain a consistent set of base conditions through the screening and alternative development process during CEPP, the base condition assumptions were not modified through the CEPP formulation process, although corrections were incorporated as necessary through periodic updated releases of the base conditions throughout the formulation process. Notably, SFWMD modeling updates for the operational assumptions of the A-1 FEB, STA-2, and STA-3/4 within the SFWMD Restoration Strategies project (included in the CEPP FWO baseline) which were developed concurrent with CEPP final array modeling (USACE draft EIS was released in February 2013, with the final EIS released in July 2013) was not incorporated into the CEPP FWO base condition assumptions. Additionally, the USACE implementation of the Everglades Restoration Transition Plan (ERTP) and associated WCA-3A Regulation Schedule changes (Record of Decision in October 2012) were not incorporated into the CEPP ECB base condition assumptions. However, following identification of the Recommended Plan Alternative 4R2, the CEPP base condition assumptions were updated to reflect best available information as of June 2013. The revised 2012 Existing Condition Baseline (2012EC) updated the ECB to include implementation of ERTP operations for WCA-3A and the South Dade Conveyance system, in addition to minor localized corrections to improve RSM-GL representation of the S-9/S-9A operations and the L-28 weir (all other ECB assumptions remain unchanged; the complete assumptions tables for the ECB and 2012EC are provided in Annex A-2, Reference 2). The revised Initial Operating Regime Baseline (IORBL1) updated the FWO to include final SFWMD proposed operational intent for the Restoration Strategies project, the 2.6 mile western Tamiami Trail bridge proposed with the initial increment of the DOI Tamiami Trail Next Steps Project (based on best available phased implementation information from DOI), operational updates to the CERP Indian River Lagoon South (IRLS) project (based on best available information from the IRLS project team), and operational refinements to the CERP Broward County Water Preserve Area project (to reduce excess discharges to tide via S-29, including accounting for the effects of the Lake Belt expansion assumed in the CEPP FWO condition), in addition to the same minor localized corrections included with the 2012EC to improve RSM-GL representation of the S-9/S-9A operations and the L-28 weir (all other FWO assumptions remain unchanged; the complete assumptions tables for the FWO and IORBL1 are provided in Annex A-2, Reference 2). The 2012EC and the IORBL1 represent the existing condition baseline and future without project baseline assumptions for purposes of completing the CEPP assessments for the Savings Clause and Project Assurances, as documented in Annex B of the main PIR report. The quantification of Redline flow volumes and timing in the Engineering Appendix therefore utilizes the 2012EC and IORBL1 base conditions as the basis for comparison to the with-project Recommended Plan Alternative 4R2, since this information reflects the best available set of assumptions and corresponds to the information analyzed in Annex B; the revised base conditions do not significantly alter the results or conclusions that are summarized in Section A.8 of the Engineering Appendix and the Hydrologic Modeling Annex A-2.

Due to the high degree of inter-annual and intra-annual variability to rainfall and other climatic parameters in South Florida, as well as the resulting differences in antecedent conditions from one year to the next, the annual surface water inflows to WCA-2A and WCA-3A are similarly variable from year to year. Total average annual water year surface water inflows to WCA-2A and WCA-3A across the Redline are summarized in Table A.8-6 for the base conditions and the Recommended Plan Alternative 4R2; for this computation, the water year was defined as May of year 1 through April of year 2 (water year is denoted as year 2). Surface water inflows along the redline to WCA-2A correspond to the sum of structure inflows from the S-7 pump station and STA-2 outflows to WCA-2A. Surface water inflows along the redline to WCA-3A correspond to the sum of structure inflows from the S-8 pump station to the Miami Canal within WCA-3A, the S-150 gated culvert, and STA-5/STA-6 outflows to northwest WCA-3A for the 2012EC and IORBL1 base conditions; for Alternative 4R2, the combined flows from the S-8 pump station discharges to the Miami Canal and discharges to the S-8A gated culvert (which diverts water to the L-4 Levee degrade gap) are included in addition to S-150 and STA-5/STA-6 outflows to WCA-3A. Water supply deliveries to the regional system from S-150 (Alternative 4R2 only), the S-8 pump station (2012EC and IORBL1 only), and the S-7 pump station (2012EC, IORBL1, and Alternative 4R2), which total a combined 44-45 kAF average annual for the 2012EC and IORBL1 and 70 kAF average annual for Alternative 4R2, were not included in the quantification of total redline flows because this volume allocated for regional water supply deliveries does not represent water made available by the CEPP project for the natural system, which is the purpose of the redline quantification for project assurances.

The average annual water year data is provided in time series and probability exceedance (volume probability curve) formats in Figure A.8-35 and Figure A.8-36, respectively. The annual water year differences (Alternative 4R2 minus 2012EC; Alternative 4R2 minus IORBL1) were rank sorted and plotted in probability exceedance format in Figure A.8-37. The intra-annual average annual water year variability across the redline is shown in Figures A.8-38 through A.8-40 for WCA-2A redline flows, WCA-3A redline flows, and total redline flows, respectively.

The L-6 diversion and associated infrastructure, which is included in Alternative 4R2, provides a means to redirect treated inflows from WCA-2A (STA-2 outflows) to northwest WCA-3A, to achieve CEPP objectives and desired depths and durations within both WCA-2A and WCA-3A. The L-6 diversion daily flow data is provided in time series and probability exceedance formats in Figure A.8-41 and Figure A.8-42, respectively.

TABLE A.8-6: AVERAGE ANNUAL FLOWS AT THE CEPP REDLINE

	Average Annual Water Year Inflows along CEPP Redline (kAF/year)		
	2012EC	IORBL1	ALT4R2
WCA-2A	329	437	290
WCA-3A	658	538	900
	Average Annual Water Year Inflows along CEPP Redline (kAF/year)		
	2012EC	IORBL1	ALT4R2
Redline total	987	976	1190
Redline total maximum	1745	1723	2074
Redline total minimum	471	473	465
Redline total std. dev.	318	319	450

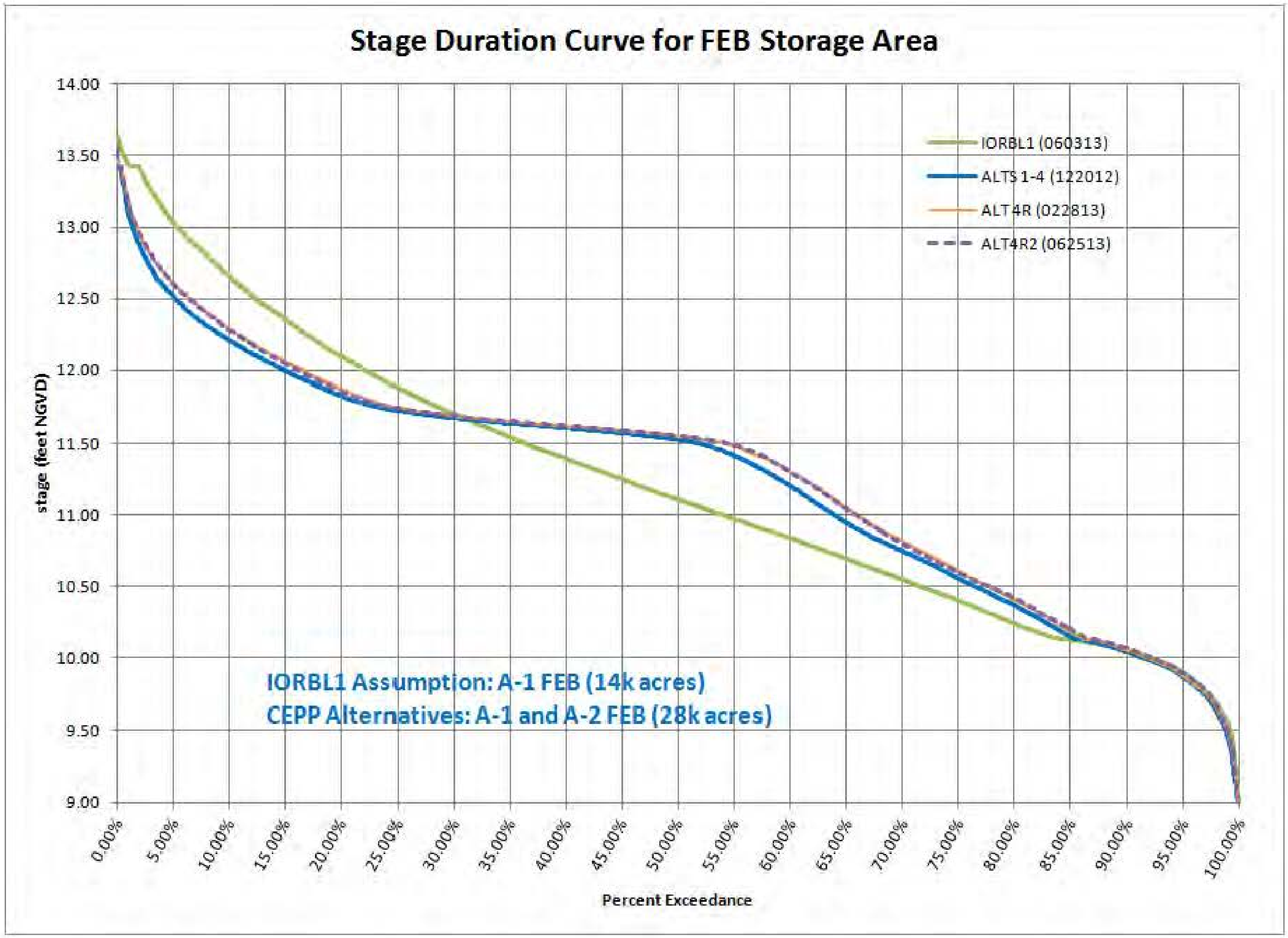


FIGURE A.8-34: FLOW EQUALIZATION BASIN STAGE DURATION CURVES FOR CEPP ALTERNATIVES 4, 4R, AND 4R2

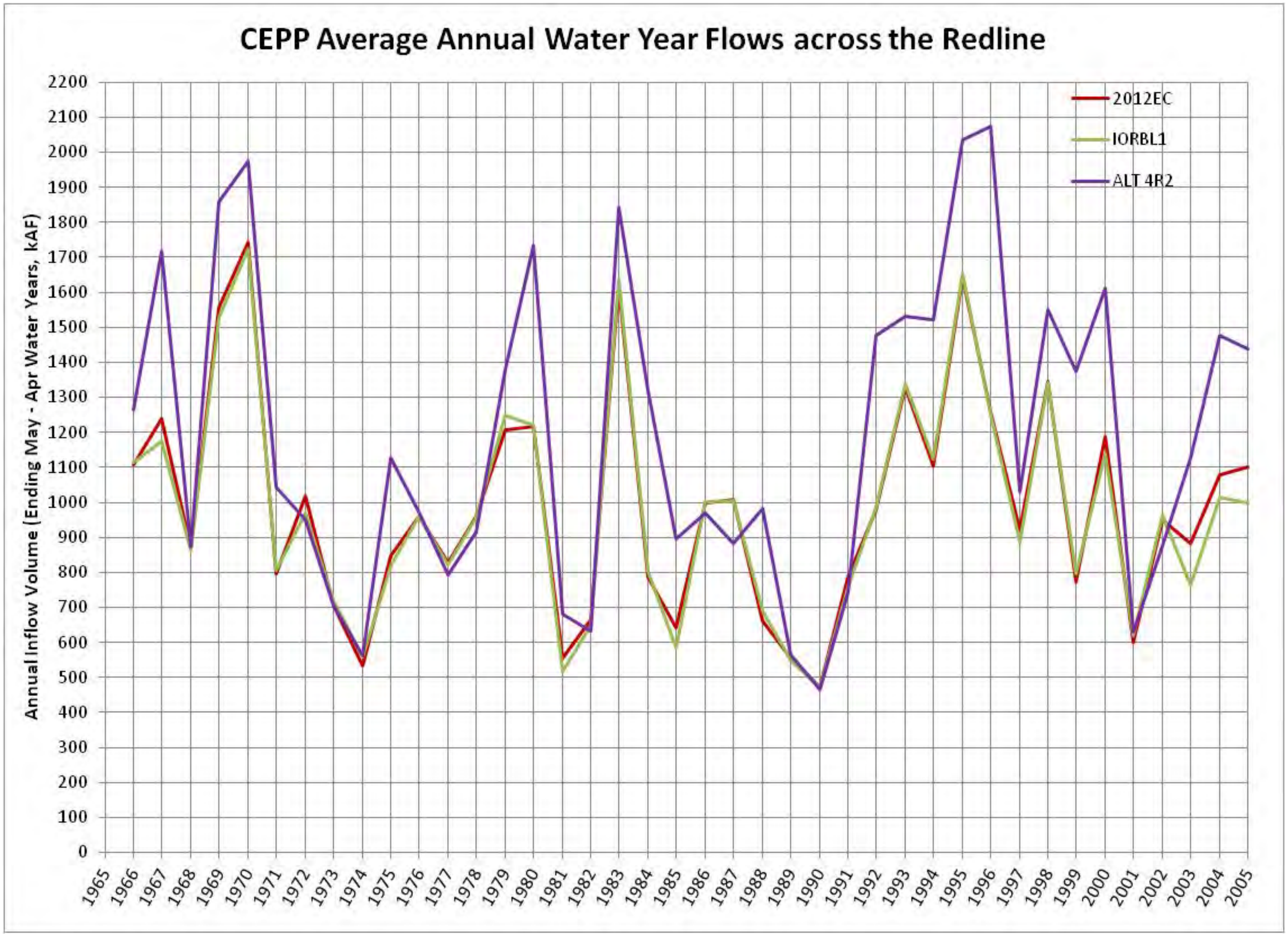


FIGURE A.8-35: ANNUAL WATER YEAR FLOWS ACROSS THE REDLINE FOR PROJECT ASSURANCES

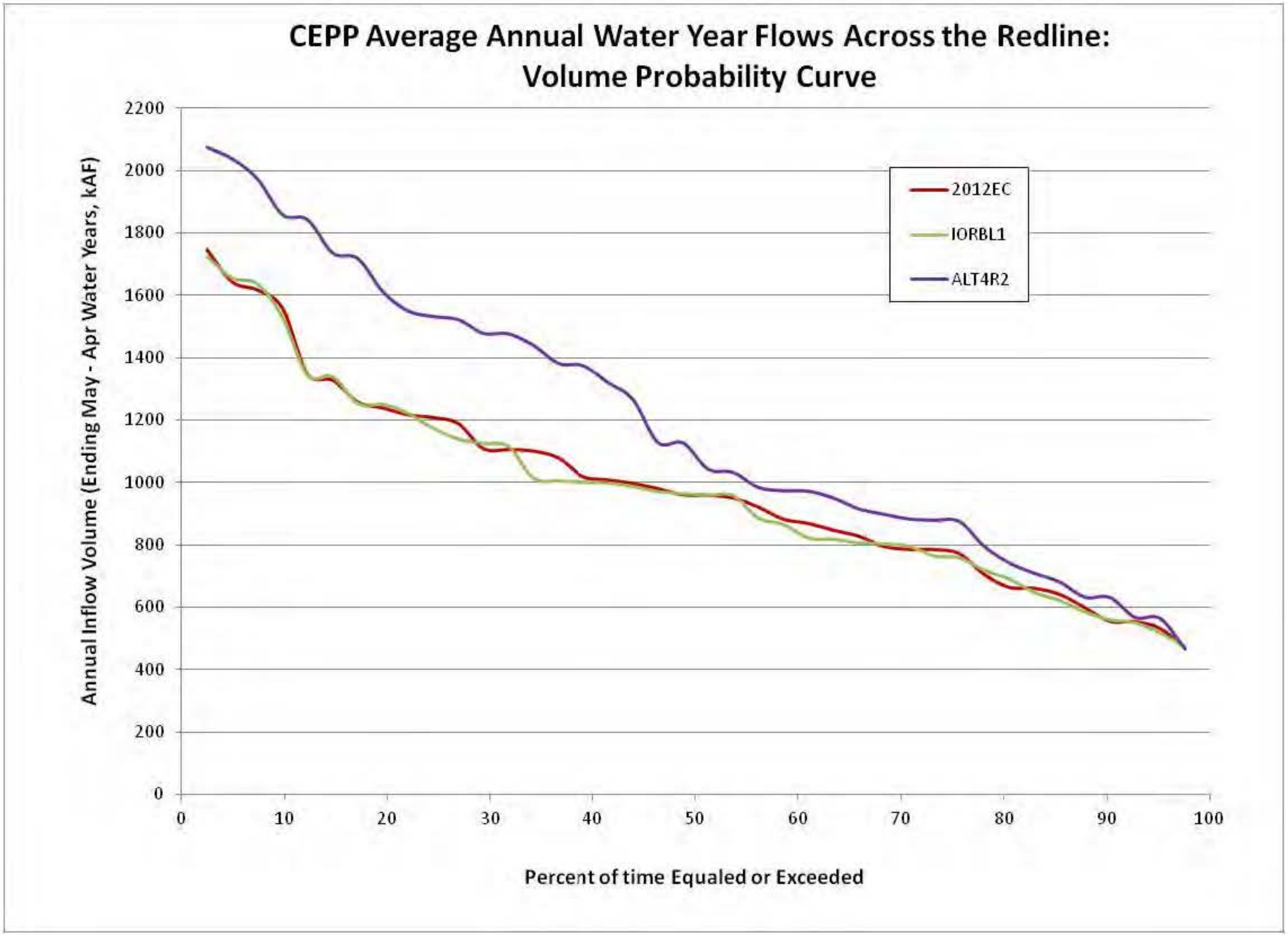


FIGURE A.8-36: VOLUME PROBABILITY CURVES OF ANNUAL WATER YEAR FLOWS ACROSS THE REDLINE FOR PROJECT ASSURANCES

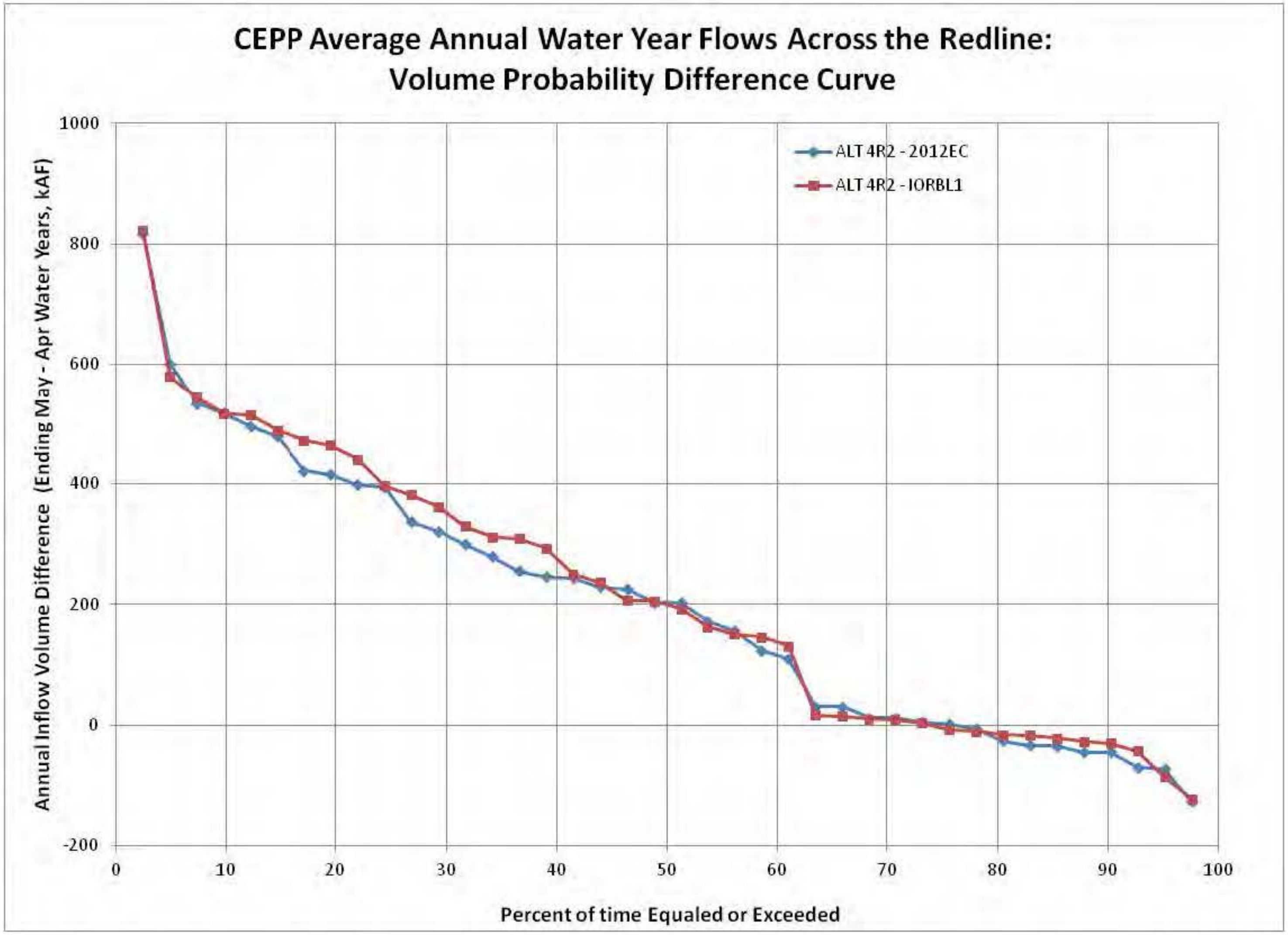


FIGURE A.8-37: VOLUME PROBABILITY CURVES OF ANNUAL WATER YEAR FLOWS ACROSS THE REDLINE FOR PROJECT ASSURANCES

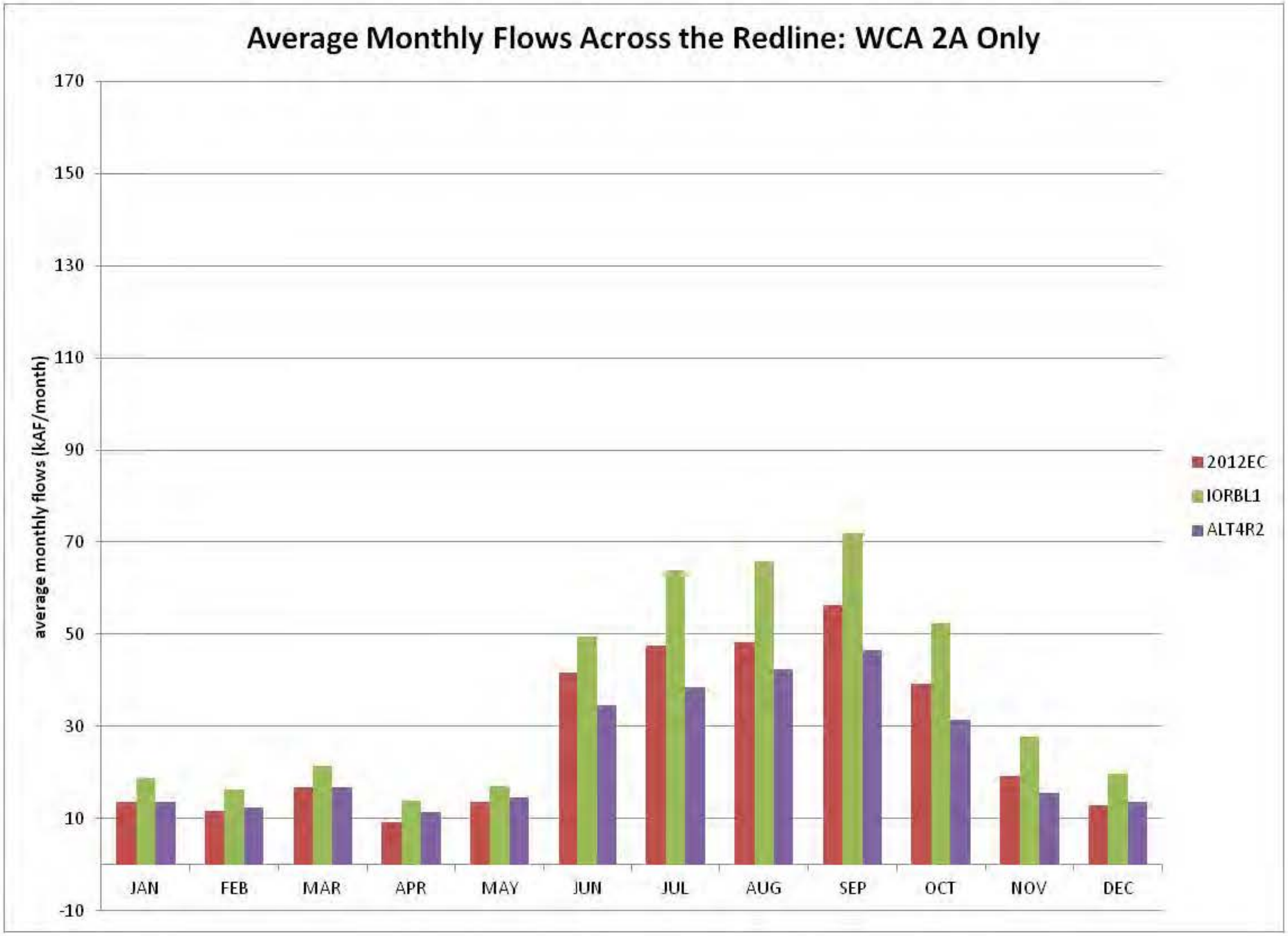


FIGURE A.8-38: INTRA-ANNUAL WATER YEAR FLOW VARIABILITY ACROSS THE REDLINE AT WCA-2A

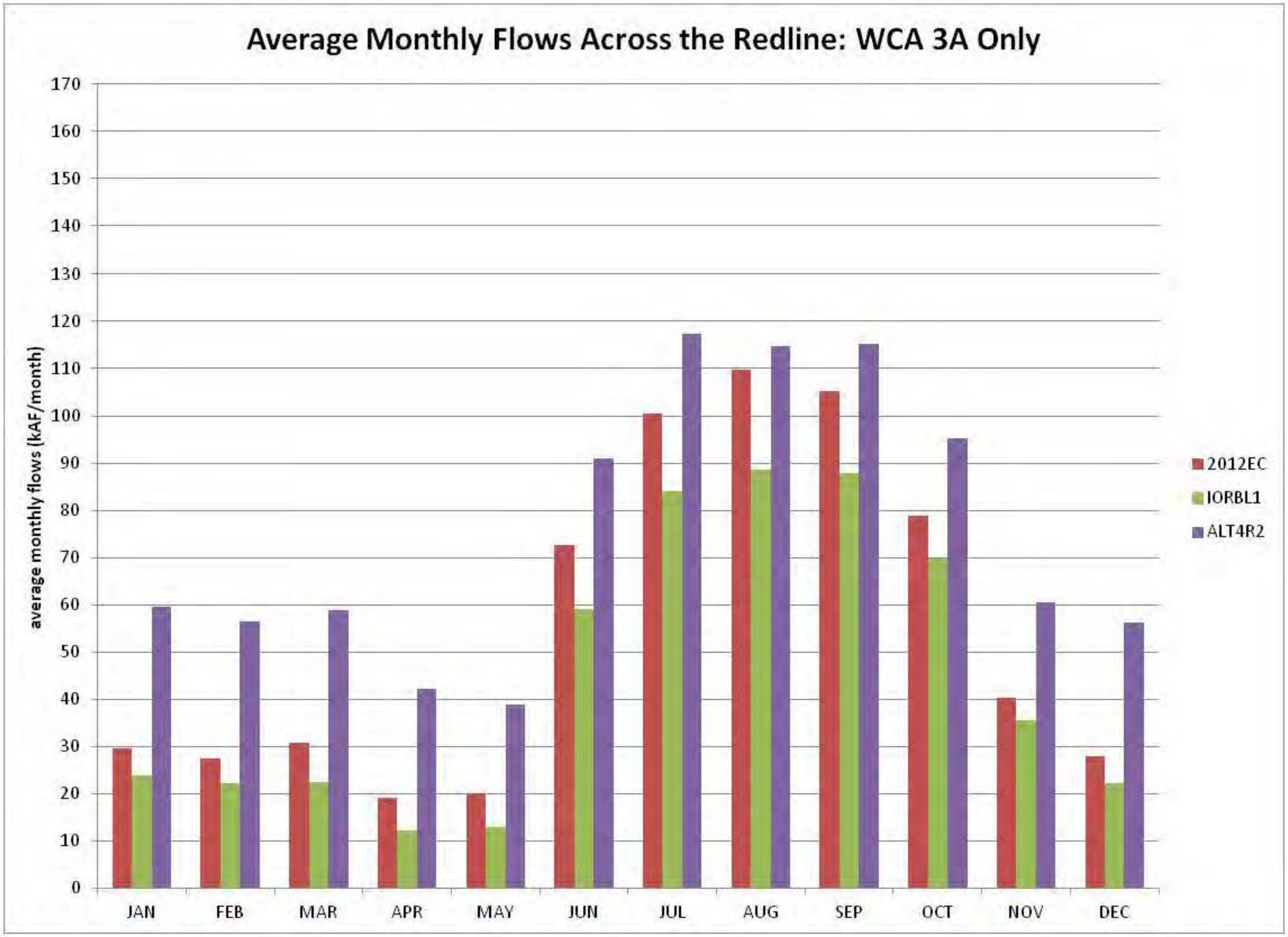


FIGURE A.8-39: INTRA-ANNUAL WATER YEAR FLOW VARIABILITY ACROSS THE REDLINE AT WCA-3A

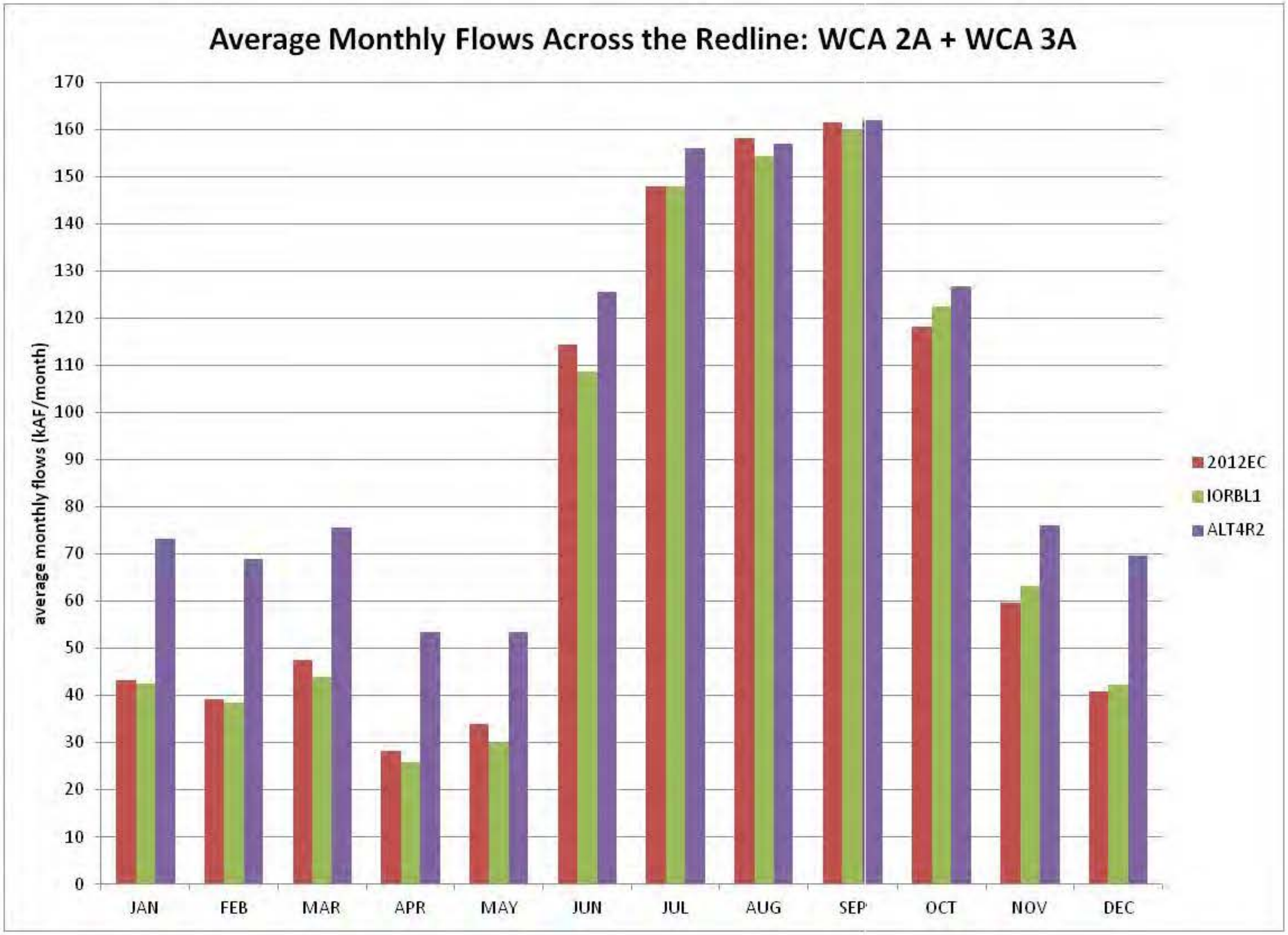


FIGURE A.8-40: INTRA-ANNUAL WATER YEAR FLOW VARIABILITY ACROSS THE REDLINE FOR WCA-2A AND WCA-3A COMBINED TOTAL

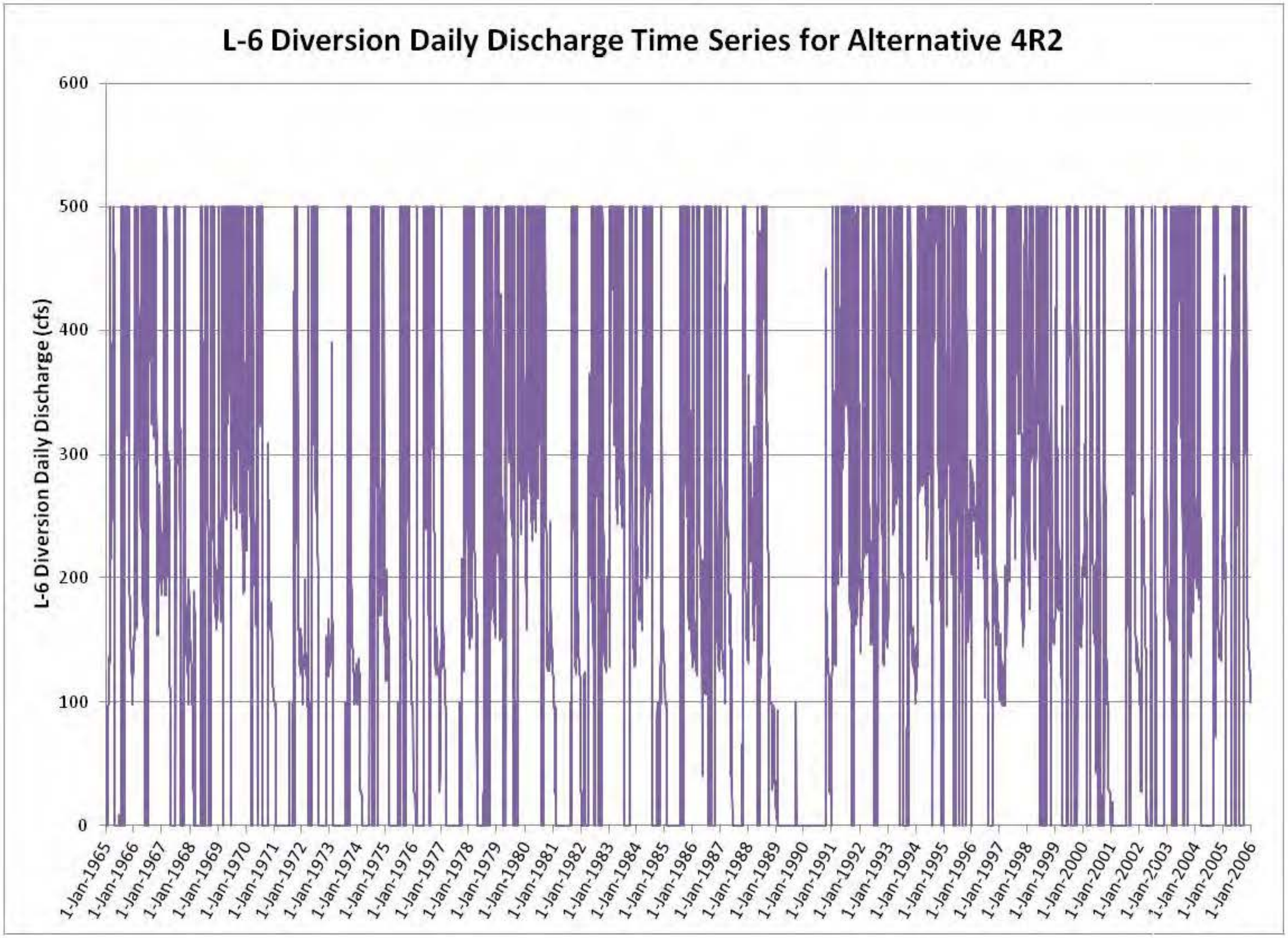


FIGURE A.8-41: L-6 DIVERSION DISCHARGE RATE TIME SERIES FOR ALTERNATIVE 4R2

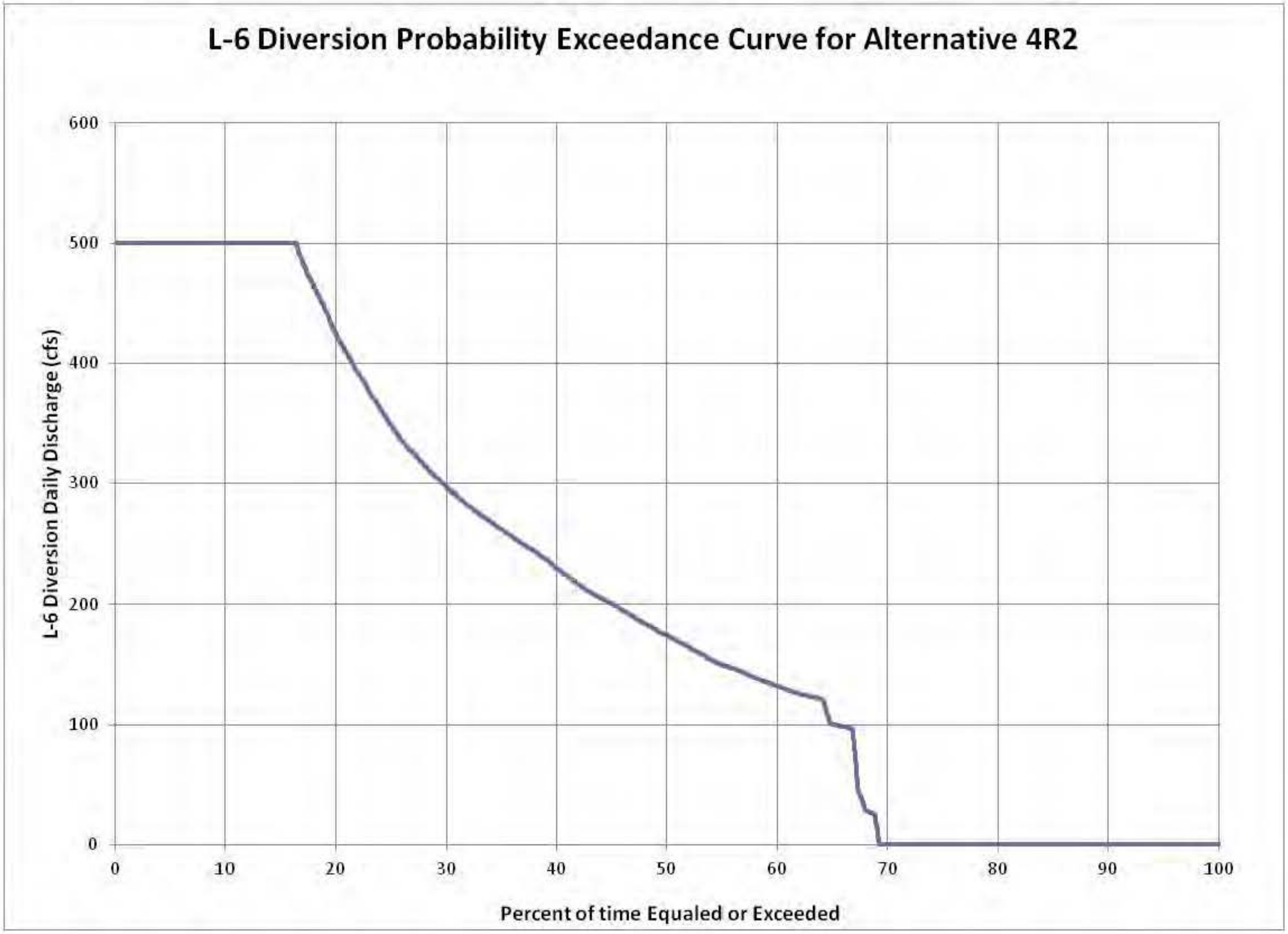


FIGURE A.8-42: L-6 DIVERSION DISCHARGE RATE PROBABILITY EXCEEDANCE FOR ALTERNATIVE 4R2

A.8.3.2.6 Quantification of Yellowline Seepage Flow Volumes

The ECB and FWO base conditions for CEPP do not include active seepage management along the L-30 Canal and L-31N Canal, which correspond to the portions of the LEC located adjacent to WCA-3B and the ENP NESRS, respectively. The ECB base condition includes active seepage management along portions of the C-111 Canal within LEC Service Area 3, east of ENP between structures S-331 and S-176, with ERTP operations of the existing S-332B, S-332C, and S-332D pump stations and the corresponding C-111 South Dade project South Detention Area (SDA) reservoirs located west of the canals. In addition to continued ERTP operations of the S-332B, S-332C, and S-332D pump stations, the FWO base condition additionally includes the following additional seepage management features: completion of the C-111 South Dade project North Detention Area reservoir, revised operations for the 8.5 Square Mile Area S-357 pump station and southern detention cell, and increased storage capacity for S-332B reservoir inflows; and completion of the CERP C-111 Spreader Canal Western PIR, including the Frog Pond Detention Area reservoir and operation of the corresponding S-199 and S-200 inflow pump stations located along the C-111 Canal between S-176 and S-177. The Recommended Plan Alternative 4R2 provides increased seepage management capability along the L-30 Canal and L-32N Canal through completion and operation of the 1000 cfs S-356 pump station (to replace the existing temporary 500 cfs S-356 pump station) and an approximately 4.2 mile long, 35 feet deep tapering seepage barrier cutoff wall along the L-31N Levee just south of Tamiami Trail and east of the ENP NESRS.

Table A.8-7 and Table A.8-8 provide a summary of the resultant seepage quantities along the L-30 Canal, L-31N Canal, and C-111 Canal. The seepage quantities are provided on an average annual basis for the 41-year RSM-GL period of simulation, using the canal sub-segments as shown on Figure A.8-43 (listed from north to south): L-30 North (L-30 north of the bridge in the tables); L-30 North of S-335 (L-30 between S-335 and the bridge in the tables); L-30 South of S-335 (same label in tables); L-31N North of G-211 (same label in tables); L-31N between G-211 and S-331 (L-31N from G-211 to S-331 in tables); L-31N South of S-331 (L-31N from S-331 to S-176 in tables); C-111 between S-176 and S-177 (C-111 from S-176 to S-177 in tables); and C-111 between S-177 and S-18C (C-111 from S-177 to S-18C in tables). RSM-GL modeling results are summarized in Table A.8-4 and Table A.8-5 for the following simulations: ECB, 2012EC, IORBL1, Alternative 4R (ALT4R), Alternative 4R2 (ALT4R2). Additional seepage quantification tables are available with the complete set of posted RSM-GL model output, to include portions of the LEC north of L-30.

TABLE A.8-7: AVERAGE ANNUAL LEVEE SEEPAGE FLOWS AT THE CEPP YELLOWLINE FOR ECB, FWO, ALTERNATIVE 4R, AND ALTERNATIVE 4R2

Annual Average
kac-ft

	Levee Seepage from Marsh Cell			
	ECB	FWO	ALT4R	ALT4R2
C-111 from S-176 to S-177	98	188	217	214
C-111 from S-177 to S-18C	29	43	49	47
L-30 between S-335 and the bridge	111	107	141	141
L-30 north of the bridge	218	214	203	201
L-30 south of S-335	92	82	98	100
L-31N from G-211 to S-331	29	29	28	28
L-31N from S-331 to S-176	209	236	329	322
L-31N north of G-211	149	164	211	251
L-31N changed to N of G-211 for 2966_3183 cell pr			38	

TABLE A.8-8: AVERAGE ANNUAL LEVEE SEEPAGE FLOWS AT THE CEPP YELLOWLINE FOR ECB, 2012EC, IORBL1, ALTERNATIVE 4R, AND ALTERNATIVE 4R2

Annual Average
kac-ft

	Levee Seepage from Marsh Cell				
	ECB	2012EC	IORBL1	ALT4R	ALT4R2
C-111 from S-176 to S-177	98	107	201	217	214
C-111 from S-177 to S-18C	29	30	44	49	47
L-30 between S-335 and the bridge	111	111	106	141	141
L-30 north of the bridge	218	215	211	203	201
L-30 south of S-335	92	92	84	98	100
L-31N from G-211 to S-331	29	29	30	28	28
L-31N from S-331 to S-176	209	207	227	329	322
L-31N north of G-211	149	171	160	211	251
L-31N changed to N of G-211 for 2966_3183 cell pr			23	38	

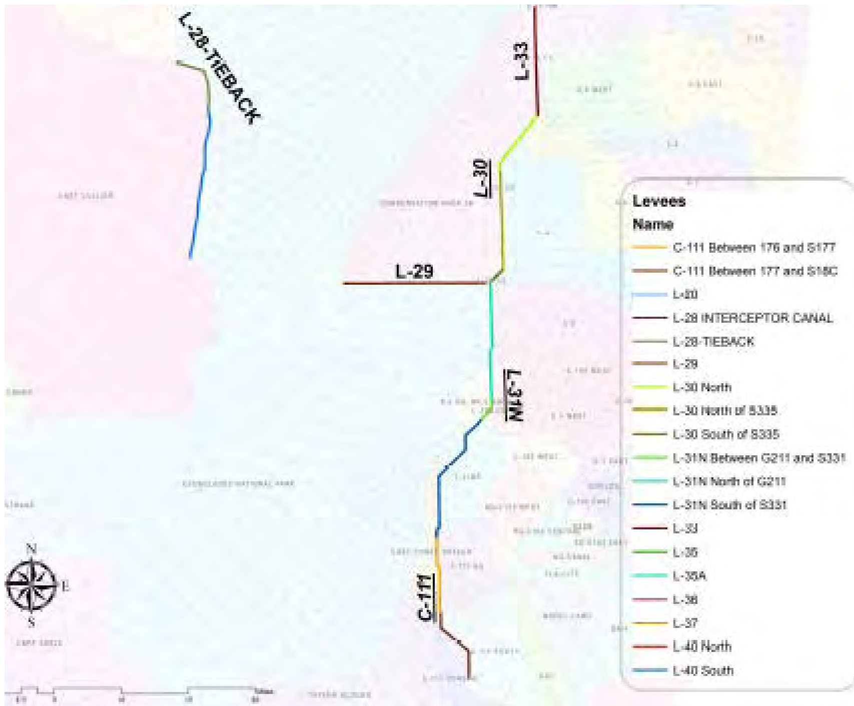


FIGURE A.8-43: RSM-GL LEVEE AND CANAL SEGMENT FOR SEEPAGE QUANTIFICATION

A.8.3.2.7 8.5 Square Mile Area Flood Mitigation Performance

The 8.5 Square Mile Area (8.5 SMA) is a primarily residential area adjacent to, but west of, the L-31N Canal. The 8.5 SMA, which is also known as the Las Palmas community, is bordered on both the west and north by NESRS (Figure A.8-44). The community has water management infrastructure consisting of a perimeter levee, a seepage collection canal, a pump station (S-357), and a southern detention cell meant to collectively provide flood mitigation as part of the MWD Project.

Stages within the 8.5 SMA, located along the eastern boundary of ENP, do not change significantly between the CEPP ECB and the FWO. The 8.5 SMA project components and operations are unchanged between the ECB and FWO modeling assumptions, with each baseline condition assuming operations of S-357 and S-331 as defined in the 2011 8.5 SMA Interim Operational Criteria; the S-357 pump station is limited to a 125 cfs average daily discharge rate, and S-331 flood mitigation operations for the 8.5 SMA are triggered based on the stage at the LPG-2 monitoring gauge (located within the protected area, along the western perimeter levee).

The CEPP alternatives modify the FWO operations of the S-357 pump station, in an effort to increase discharges from the 8.5 SMA detention cell to the C-111 South Dade North Detention Area and reduce the reliance on the S-331 pump station in L-31N to provide flood mitigation for the 8.5 SMA protected area. The protected portion of the 8.5 SMA is represented by only 3 model grid cells in the RSM-GL, and the resolution of the RSM-GL is extremely limiting for adequate representation of the 8.5 SMA project features. Prior to implementation of CEPP, further technical investigations and potentially additional hydrologic/hydraulic modeling with a higher resolution model will likely be needed for the 8.5 SMA operations. The current MWD 8.5 SMA configuration was identified in the USACE C&SF MWD 8.5 SMA General Reevaluation Report (2000 GRR), which provided a detailed quantification of potential affects to 8.5 SMA flood mitigation performance and potential affects to adjacent ENP wetlands supported by ModBranch hydrologic modeling.

RSM-GL final array modeling of Alternatives 1 through 4 indicated that stages within the 8.5 SMA were lowered by approximately 0.25 feet during wet conditions for the northern and southeastern areas of the 8.5 SMA, compared to the FWO. However, of concern with Alternatives 1 through 4, stages within the southwest portion of the 8.5 SMA were increased by approximately 0.3-0.6 feet, compared to the FWO, under all hydrologic conditions. These alternatives maintained increased utilization of the S-357 pump station to provide effective flood mitigation for the 8.5 SMA protected area but did not include lowering of the overflow weirs' elevations within the 8.5 SMA detention area (crest elevations for the S-360W and S-360E weirs were maintained at the elevations specified for the 2011 Interim Operations Plan for 8.5 SMA, corresponding to overflow depths of 4.0 and 3.5 feet, respectively); consistent with previous field observations during S-357 interim operations, the CEPP modeling demonstrated that increased operational depths within the 8.5 SMA detention area may potentially cause increased groundwater stages within the southwestern portion of the 8.5 SMA protected area. Stage duration curve graphics for Alternatives 1 through 4R2 are further described and included in the Hydrologic Modeling Annex A-2.

The 8.5 SMA detention cell weirs were lowered with Alternative 4R and Alternative 4R2 to allow overflow when depths exceeded 1.0 feet, which resulted in performance improvements within the southwestern portion of the 8.5 SMA protected area. RSM-GL modeling of the Recommended Plan Alternative 4R2 indicates that stages within the 8.5 SMA are lowered by approximately 0.25-0.50 feet during wet conditions for the three RSM-GL grid cells that represent the protected portion of the 8.5 SMA, compared to the FWO.

FIGURE A.8-44: LOCATION MAP FOR 8.5 SMA

A.8.3.2.8 Additional RSM-GL Post-Processing for Structures and Detention Areas

RSM-GL daily output for structure discharges and water stages at monitoring gauges are generated for the 1965-2005 period of simulation and tabulated using the USACE Hydrologic Engineering Center's Data Storage System (HEC-DSS). Due to the enormous volume of data included in the RSM-GL DSS files for the CEPP baselines and the CEPP alternatives, EN-W developed an additional suite of post-processed RSM-GL graphics to facilitate review of the preliminary Blue Line and Yellow Line screening modeling and the final array modeling by the CEPP water supply and flood control (WS/FC) technical sub-team. The primary assessment focus of the CEPP WS/FC sub-team was the South Dade Conveyance System (SDCS), including the effects of controlled/uncontrolled increased seepage from WCA-3B and eastern ENP with implementation of CEPP components and operations; the seepage flux dynamics along the Yellow Line are directly correlated to increased flood control risk (too much increased seepage and/or too little active seepage management) and reduced water availability for water supply (too little increased seepage and/or too much active seepage management).

Using the list of critical flow structures that was identified by EN-W for CEPP and included in the average annual critical flows reports, flow duration curve graphics were generated by EN-W for each of these critical structures to quantify the degree to which existing and/or proposed structure design capacities are sufficient for achievement of CEPP objectives, as well as the relative differences between the screening simulations and final alternatives. Stage duration curve graphics were also generated by EN-W for the 8.5 SMA Detention Area, C-111 North Detention Area, C-11 South Detention Area, and the Frog Pond Detention Area, to assess the relative differences in utilization of these storage areas for which standard model output graphics were not otherwise available. Several of the EN-W flow duration curves and stage duration curves were particularly utilized by the CEPP WS/FC sub-team during sub-team review of the final array modeling, and a selected sub-set of these graphics are provided in the Hydrologic Modeling Annex A-2. Aside from the unprocessed DSS output files, these flow duration curve and stage duration curve graphics are not otherwise available in the posted RSM-GL standard model output.

A.8.4 Identification of Additional Hydrologic Modeling for PED

Although the RSM-BN and RSM-GL hydrologic models are well suited to support the preliminary screening, alternative formulation, and evaluation of CEPP alternatives, it is expected that higher resolution hydrologic and hydraulic modeling tools will be required to further analyze localized and possibly regional-scale effects of specific components of the CEPP Recommended Plan, with the scope of these analyses further identified during the Pre-Construction Engineering and Design (PED) phase of the project. Based on components currently identified for the CEPP Recommended Plan, the following provides a minimum list of project components likely to require further hydrologic and hydraulic analyses and/or modeling during PED:

1. Identification of S-8 Pump station modifications (or potential replacement) required to redirect most S-8 outflows west to the L-4 Canal: An undefined length of the Miami Canal downstream of S-8 (1-2 miles is currently assumed) may be required to provide hydraulic conveyance under peak flood events, while maintaining S-8 tailwater conditions within the design criteria range of the S-8 pump.

2. Determination of the effects of the L-67 Extension Levee and Canal removal on the discharge capability of the S-12 spillways, including consideration of removal of all or portions of the old Tamiami Trail roadway located in close proximity to the S-12C and S-12D outlets.

3. Identification of potential conveyance improvements within the WCA-3B remnant agricultural ditches, east of the Blue Shanty levee (L-67D) to improve the ability to achieve north-to-south flows from eastern WCA-3B to the L-29 Canal, via the existing S-355A and S-355B gravity spillway structures. For example, by providing a hydrologic connection between the S-355B collector canal and the remnant agricultural ditches, the efficiency and quantity of conveyance from southern WCA-3B to the S-355B will be improved.

4. Determination of the required length and depth for the L-31N seepage cutoff wall to achieve the desired balance between seepage management, flood control, and water supply objectives, including potential consideration of monitoring data from similar existing seepage cutoff wall features and the potential need for additional design tests to support detailed design. Design capacity for the S-356 seepage management pump station may also be affected by design changes with the seepage wall.

5. Identification of structure operations within the South Dade Conveyance System during phased implementation of CEPP project components, including detailed operational planning studies that were not generally beyond the scope of CEPP formulation efforts.

6. Further technical investigations and potentially additional hydrologic/hydraulic modeling with a higher resolution model will likely be needed for the 8.5 SMA operations. The current MWD 8.5 SMA configuration was identified in the USACE C&SF MWD 8.5 SMA General Reevaluation Report (2000 GRR), which provided a detailed quantification of potential affects to 8.5 SMA flood mitigation performance and potential affects to adjacent ENP wetlands supported by ModBranch hydrologic modeling.

Additional data and analysis needs will also likely be identified during PED phase.

A.9 OPERATIONS AND MAINTENANCE

Operation, Maintenance, Repair, Replacement and Rehabilitation (OMRR&R) begins after project construction and Operational Testing and Monitoring is complete and generally includes all operation activities and maintenance needed to keep the project features functioning as intended. OMRR&R for the CEPP project will occur for all new facilities constructed as a result of the project, and as an increase to the OMRR&R for State Facilities that CEPP will use to provide new water to the WCAs and ENP. There will be OMRR&R for CEPP features and for State facilities used by CEPP. Provided below are the tables for CEPP Project features and State facilities used by CEPP.

Activities included in the OMRR&R costs are:

- Pump and facility maintenance which are per manufacturer's recommendations and schedules.
- Repair and rehabilitation of pumps, drivers, and switchgear are assumed to be rehabilitated or replaced once during the 50-year life.
- Erosion control to make sure banks and areas around culverts and other structures are not compromised by weather, plant or animal forces.
- Mowing to maintain grass areas for a neat and clean appearance and also to make sure there are no other maintenance issues being hidden by high grass vegetation. Mowing also reduces the ability of woody plants to gain a foothold and lead to larger issues.
- All monitoring, required by permit, USFWS Incidental Take Statement, and/or needed to adaptively manage the Project.

- Invasive, exotic, native, and nuisance vegetation control. Vegetation control is done both to control underwater infestations and surface infestations. Invasive plants can prevent correct project function and can damage vital structural components if allowed to grow unchecked.
- Adaptive Management (AM) measures needed to ensure project benefits or avoid violating one or more project constraints.

A.9.1 CEPP project features.

Structure	OMRR&R Costs
A-2 FEB	$2,090,000
S-620 (CS-1) 500 cfs gated culvert, S-621 (CS-2) 2500 gated spillway, S-622 (CS-3) 500 cfs gated culvert	$330,000
Modified S-8 (2 gated culverts)	$230,000
S-630 (360 cfs PS)	$240,000
New S-333N - 1150 cfs	$160,000
New (S-356) PS at 1000 cfs	$600,000
500 cfs gated structures (S-631, S-632, and S-633)	$340,000
8.5 mile levee in WCA 3B	$50,000
S-355W-1230 cfs gated structure	$110,000
TOTAL Average Annual OMRR&R Costs New Facilities	**$4,150,000**

A.9.2 State facilities used by CEPP

The future OMRR&R costs of operating the system without CEPP features once CEPP is constructed and operational is based on new water flows through the state facilities as a portion of the overall water flows through the state facilities. Reference 6.4.2 of the main PIR for cost sharing information.

TABLE A-35. LIST OF STA 3/4 AND ASSOCIATED INFRASTRUCTURE (1)

Structure	Structure/Design Capacity	Description
G-370	2775 cfs Pump	Inflow Pump
G-370S	225 cfs Pump	Seepage Return Pump
G-371	2170 cfs Gated Spillway	North New River Canal Divide Structure (used for STA diversions and water supply)
G-372	3700 cfs Pump	Inflow Pump
G-372HL	250 cfs Gated Culvert	Culvert to convey untreated stormwater or STA-3/4 seepage to Holey Land WMA
G-372S	225 cfs Pump	Seepage Return Pump
G-373	2400 cfs Gated Spillway	Miami Canal Divide Structure (used for STA diversions and water supply)
G-374 A, B, C, D, E and F	362 cfs Gated Culvert	Cell 1A Inflow Structure
G-375 A, B, C, D, E and F	362 cfs Gated Culvert	Cell 1B Inflow Structure
G-376 A, B, C, D, E and F	362 cfs Gated Culvert	Cell 1B Outflow Structure
G-377 A, B, C, D, and E	396 cfs Gated Culvert	Cell 2A Inflow Structure
G-378 A, B, C, D, and E	396 cfs Gated Culvert	Cell 2B Inflow Structure

G-379 A, B, C, and D	396 cfs Gated Culvert	Cell 2B Outflow Structure
G-379E	396 cfs Gated Culvert	Lower SAV Cell Outflow Structure (PSTA)
G-380 A, B, C, D, E and F	282 cfs Gated Culvert	Cell 3A Inflow Structure
G-381 A, B, C, D, E and F	282 cfs Gated Culvert	Cell 3B Outflow Structure
G-382A	Various cfs Gated Culvert	Cell 1A/Cell 2A Transfer Structure
G-382B	Various cfs Gated Culvert	Cell 2A/Cell 3B Transfer Structure
G-383	1470 cfs Gated Culvert	Inflow Canal Divide Structure
G-384 A, B, C, D, E and F	282 cfs Gated Culvert	Cell 3B Inflow Structure
G-385	54 cfs Pump	Cell 1A/1B Transfer/Hydration Pump
G-386	29 cfs Pump	Cell 2A/2B Transfer/Hydration Pump
G-387	24 cfs Pump	Cell 3A/3B Transfer/Hydration Pump
G-388	160 cfs Pump	Outflow Pump (PSTA)
G-389A, B	105 cfs Ungated Culvert	Lower SAV Cell Inflow Structure (PSTA)
G-390A	105 cfs Gated Culvert	PSTA Cell Inflow Structure (PSTA)
G-390B	40 cfs Gated Culvert	PSTA Cell Inflow Structure (PSTA)
G404	600 cfs Pump	Pump to convey STA-3/4 (and STA-5/6) discharges and water supply to WCA-3A
G409	190 cfs Pump	Pump to convey water supply to the Big Cypress Seminole Indian Reservation (in conjunction with G-404)
S-150	1000 cfs Gated Culvert	Culvert to convey STA-3/4 discharges and water supply to WCA-3A
S-7	2490 cfs Pump	Pump to convey STA-3/4 discharges and water supply to WCA-2A
S-8	4160 cfs Pump	Pump to convey STA-3/4 (and STA-5/6) discharges and water supply to WCA-3A

(1) STA associated infrastructure will be identified prior to executing the PPA New Water

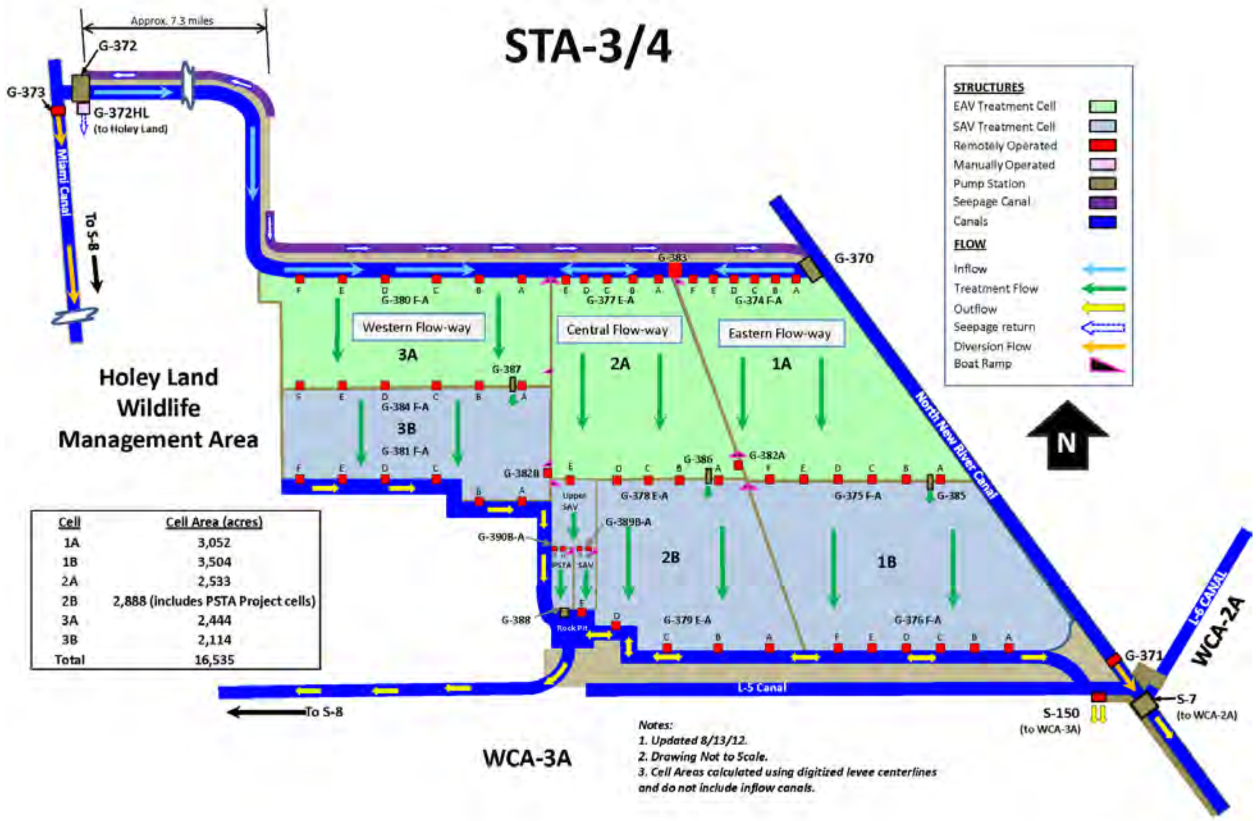


FIGURE A-9. STA 3/4 INFRASTRUCTURE

TABLE A-36. LIST OF STA 2 & ASSOCIATED INFRASTRUCTURE (1)

Structure	Structure/Design Capacity	Description
G-328	444 cfs Pump	Inflow Pump (co-located with culvert and 111 cfs pump for irrigation)
G-329A, B, C and D	197 cfs Gated Culvert	Cell 1 Inflow
G-330A, B, C , D and E	158 cfs Weir/Culvert	Cell 1 Outflow
G-331A, B, C , D, E, F and G	212 cfs Gated Culvert	Cell 2 Inflow
G-332	1485 cfs Gated Spillway	Cell 2 Outflow
G-333A, B, C , D and E	214 cfs Gated Culvert	Cell 3 Inflow
G-334	1071 cfs Gated Spillway	Cell 3 Outflow
G-335	3040 cfs Pump	Outflow Pump
G-336A, B, C , D, E, F and G	300 cfs Ungated Culvert	Culvert between L-6 Canal and WCA-2A
G-337	240 cfs Pump	Seepage Return Pump
G-337A	1020 cfs Gated Culvert	Inflow Canal Divide Structure
G-338	975 cfs Gated Spillway	Structure between Inflow Canal and WCA-1
G-339	2000 cfs Gated Spillway	Structure between Inflow Canal and L-6 Canal
G-341	600 cfs Gated Spillway	Ocean Canal Divide Structure

G-367 A, B, C , D, E, and F	110 cfs Gated Culvert	Cell 4 Inflow
G-368	1120 cfs Gated Culvert	Cell 4 Outflow
G-434	1120 cfs Pump	Inflow Pump
G-434S	300 cfs Pump	Seepage Return Pump
G-435	480 cfs Pump	Inflow Pump
G-436	1600 cfs Pump	Outflow Pump
G-338A, B, C , D and E	118 cfs Gated Culvert	Cell 5 Inflow
G-438F, G, H, I and J	106 cfs Gated Culvert	Cell 6 Inflow
G-440 A, B, C , D, E, and F	80 cfs Gated Culvert	Cell 7 Inflow
G-441	960 cfs Gated Culvert	Cell 8 Outflow
G-442	480 cfs Ungated Culvert	Cell 7 Outflow
G-443A and B	232 cfs Gated Culvert	Cell 4 Inflow
G-445	27 cfs Pump	Seepage Return Pump
S-6	2925 cfs Pump	Inflow Pump

(1) STA associated infrastructure will be identified prior to executing the PPA New Water

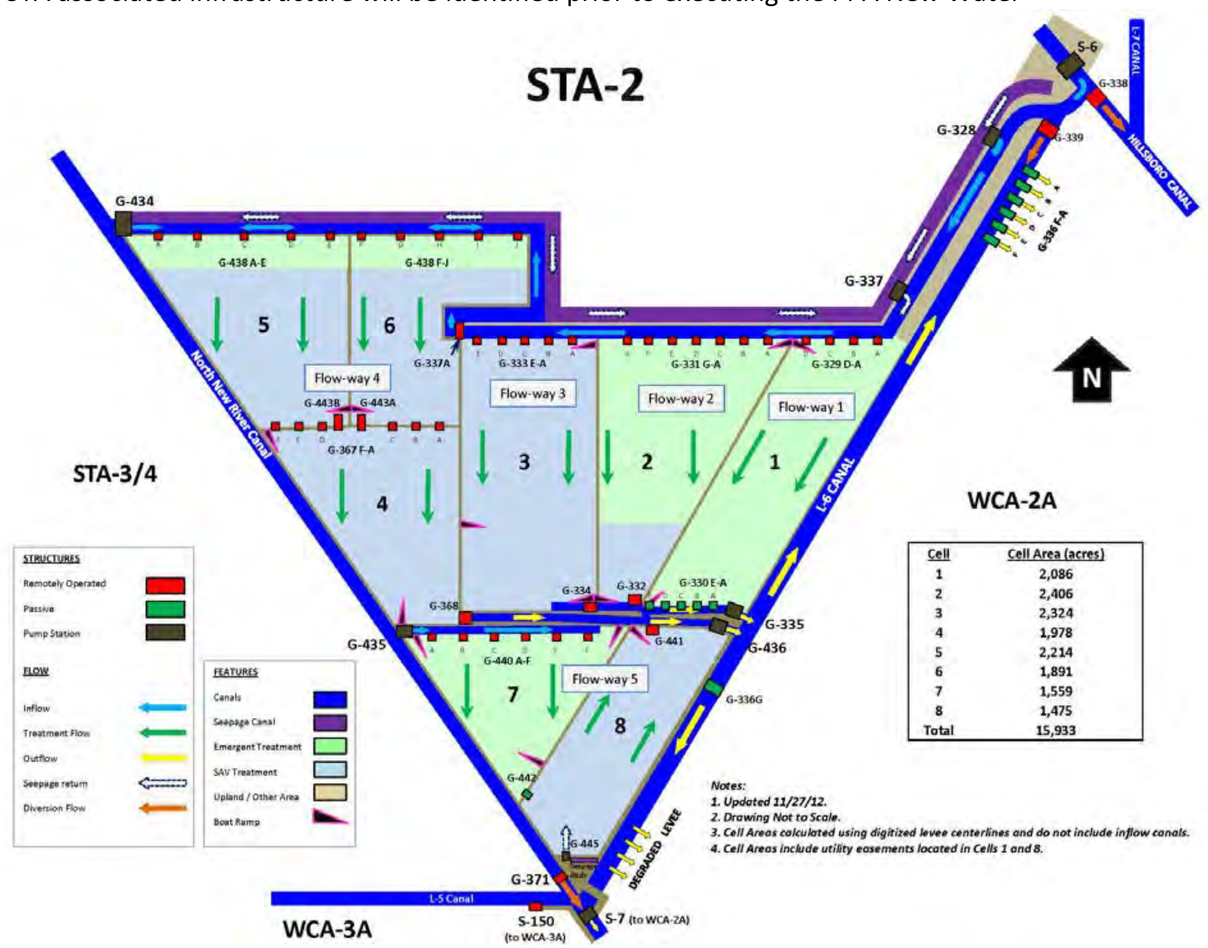


FIGURE A-10. STA 2 INFRASTRUCTURE

A.10 VALUE ENGINEERING

A joint Value Engineering (VE) Study and Cost Schedule & Risk Analysis Workshop for CEPP was conducted during the Analysis Phase 4-8 February, 2013. During the workshop various items were discussed and either screened for further consideration, retained as a possible Adaptive Management consideration or retained for Value Engineering recommendation for PED consideration. Those documented outcomes are described in the Value Engineering Study Report which is included as an Annex to Engineering Appendix and posted into the Value Engineering Library located on the USACE Value Engineering/Value Management Community of Practice Portal. A listing of the recommendations is provided below by project area. The Value Engineering Study satisfies the decision document VE requirements as stipulated in ER 11-1-321 Army Programs Value Engineering dated 01 January 2011. Appendix A, Annex B provides a copy of the Value Engineering Report.

A.10.1.1 North of Redline

- NR-1. Add outflow gravity structure on SE corner of A-2 - *Address recommendation in project design phase.*
- NR-2. Add in-line structure for North New River Canal - *Address recommendation in both current plan development and project design phase.*
- NR-3. Increase DS-8 gate capacity from 1,500 to 3,750 cfs to maintain existing drainage flowrate and water elevation - *Address recommendation in current plan development.*

A.10.1.2 South of Redline

- SR-1. Add AM strategy for G-336G (L-6 Diversion) - *Address recommendation in both current plan development and development of the adaptive management activities.*
- SR-2. Increase S-8 existing pump station horsepower and/or add supplemental exterior type pump unit(s) in lieu of constructing a new pump station - *Address recommendation in project design phase.*
- SR-3. Add new pump station (S-8) - *Address recommendation in project design phase.*
- SR-4. Integrate and optimize S-8 and G-404 system - *Address recommendation in both current plan development and project design phase.*
- SR-5. Re-visit USFWS/FWC Draft Ecological Guidelines for Water Management in WCA-2A – *Address recommendation in development of adaptive management activities.*

A.10.1.3 Blue/Green/Yellowline

- GB-1. Consider partial removal of the remaining length of the L-67 Extension Levee and/or system; also consider only partial removal of Old Tamiami Trail - *Address recommendation in development of adaptive management activities.*
- GB-2. Consider extending S-355B collector canal - *Address recommendation in both current plan development and development of the adaptive management activities*
- GB-3. Modify the ag canals in flowway - *Address recommendation in both current plan development and development of the adaptive management activities*

- GB-4. Use vegetation management to reduce vegetative resistance to water flow downstream of L-67A new structures S-345D & G - *Address recommendation in development of the adaptive management activities*
- GB-5. Retrofit DPM structure (S-152 800 cfs 10 - 60" HDPE barrels); Use DPM structure for interim period - *Address recommendation in both project design phase and development of the adaptive management activities*
- GB-6. Re-visit L-29 gated divide structure to determine actual flow need and gate flow size - *Address recommendation in both current plan development and development of the adaptive management activities*
- GB-7. Optimize operations at most northern structure into WCA 3B (consider for other control structures) - *Address recommendation in development of the adaptive management activities*
- Y-1. Determine new S-356 pump station capacity based on functional risk; do not design for both full contingency and unit redundancy - *Address recommendation in both current plan development and project design phase.*
- Y-2. For S-356, eliminate redundant pump but incorporate possible future expansion - *Address recommendation in project design phase.*
- Y-3. Defer construction of new S-356 pump station until adjacent seepage wall is constructed and system tested; further utilize existing S-356 temporary pump station - *Address recommendation in current plan development, project design phase and in development of the adaptive management activities.*
- Y-4. Phase implementation of seepage control features; use AM to determine path - *Address recommendation in development of the adaptive management activities*
- Y-5. Change the location L31N Seepage Management Pilot Project (SMPP) to the location which was the original location contained in the authorized decision document; use CEPP to increase the 902 Limit for L31N SMPP and install the L31N SMPP to remove project uncertainties. – *Not adopt. The L31N SMPP was the pilot component of the original CERP L-31N Improvements for Seepage Management. The CEPP PDT has decided to use the monitoring and results of a nearby constructed non-federal seepage project that was installed as described in main PIR sections 2.5.12 and section 6.10.2.1 to address project uncertainties for CEPP*.
- Y-6. Investigate alternative seepage barrier cutoff wall means (such as vinyl sheet pile) - *Address recommendation in project design phase.*

A.10.1.4 General

- GC-1. Create 'environmental friendly' conveyance channels where opportunity exits - *Address recommendation in both current plan development and project design phase.*
- GC-2. Coordinate vegetation management to achieve multiple objectives - *Address recommendation in development of adaptive management activities.*
- GC-3. Optimize pump station design - *Address recommendation in project design phase.*

A.10.1.5 Additional Value Engineering PED Considerations (Post VE Workshop)

After the VE workshop, cost and scope items were discussed and screened for further PED consideration. The robust description of cost assumption refinements are captured in Appendix B – Cost Engineering, Cost Assumptions documentation. A list of additional PED considerations from this effort is captured below:

- S-623 investigate construction in dry offset and offline from existing canal network and connecting both Miami Canal and STA 3 / 4 supply canal by 45 degree angle to this structure. This is to minimize interference with G-372 operations as partially restricting upstream conveyance is not preferred.
- S-628 investigate need for structure and it's function in PED. Possibly consider gravity outflow.
- L-624 investigate use of 7.5 miles of existing levee and it's capability to meet CEPP needs.
- S-621 investigate construction sequence and necessity of structure with S-622 and S-620 in coordination with STA 3 / 4 operations.
- S-8A New culverts investigate G-404 mods, S-8 mods, resizing spreader and a weir and all components tied to diverting flows south and east.
- S-622 investigate necessity for structure in PED.
- S-626 investigate tying in and utilizing the existing G-372 seepage pumps to reduce design capacity and move S-626 northward.
- C-624 investigate size and overland flow component to minimize having excess material
- C-624E investigate size and depth, remodel to minimize having excess material.
- C-626 investigate remodel and redesign cross section to minimize excess material.
- Tie A-2 FEB collection canal to the A-1 FEB to simplify canal improvements, reduce number of structures required for A-2 FEB outflow.

A.11 REFERENCES

Black and Veatch, 2006, EAA Reservoir A-1, Geotechnical Data Report, 3 Vols.

Bottcher, A.B., 1994, The EAA: Water, Soil, Crop, and Environmental Management, 310 pp.

Challenge Engineering and Testing, 2006, Final Report L-30 Seepage Management Pilot Project, Water Conservation Area 3B, Miami-Dade County, Florida.

Cunningham, K.J. and others, 2006, A cyclostratigraphic and borehole geophysical approach to development of a three-dimensional conceptual geologic model of the karstic Biscayne aquifer,

southeastern Florida: U.S. Geological Survey Scientific Investigations Report 2005-5235.

Fischer, J.N., 1980, Evaluation of a cavity-riddled zone of the shallow aquifer near Riviera Beach, Palm Beach County, Florida:U.S. Geological Survey Water-Resources Investigations Report 80-60, 39 p.

Fish, J.E., 1988, Hydrogeology, Aquifer Characteristics, and Ground-Water Flow of the Surficial Aquifer System, Broward County: U.S. Geological Survey Water-Resources Investigations Report 87-4034, 92 p, 10 pls.

Fish, J.E., and Stewart, Mark, 1991, Hydrogeology of the surficial aquifer system, Dade County, Florida: U.S. Geological Survey Water-Resources Investigations Report 90-4108, 50 p, 11 pls. Accessed at http://sofia.usgs.gov/publications/wri/90-4108/. Accessed on 19 March 2013.

Gleason, P.J. and Spackman, W., 1974, Calcareous periphyton and water chemistry in the Everglades in P. Gleason, Ed., Environments of South Florida, Past and Present, Miami Geological Society Memoir 2: 146-181.

Gleason, P.J. and others, 1984, The environmental and saline tidal plain: in P.J. Gleason, ed., Environments of South Florida, Present and Past II: Miami Geological Society, p. 297-351.

Nodarse and Associates, 2000, Stormwater Treatment Area No. 3 and 4 East WCA-3A Hydropattern Restoration L-5 Canal, Boring Profiles.

Petersen, M. and others, 2008, Documentation for the 2008 Update of the United States National Seismic Hazard Maps: U.S. Geological Survey Open-File Report 2008-1128, 61 pp.

Reese, R.S. and Wacker, M.A., 2009, Hydrologic and Hydraulic Characterization of the Surficial Aquifer System, and Origin of High Salinity Groundwater, Palm Beach County, Florida: U.S. Geological Survey Scientific Investigations Report 2009-5113, 83 p.

Renken, R.A. and others, 2013, Impact of Anthropogenic Development on Coastal Ground-Water Hydrology in Southeastern Florida, 1900-2000: U.S. Geological Survey Circular 1275, 77 p.

Shine, M.J., Padget, D.G.J., and Barfknecht, W.M., 1989, Ground water resource assessment of Palm Beach County, Florida: South Florida Water Management District Technical Publication 89-4, pt. I, 372 p.

Stephens, J.C. and Johnson, L., 1951, Subsidence of organic soils in the upper Everglades region of Florida, Soil Science Society of Florida Proceedings, v. XI, p. 191-237.

U.S. Army Corps of Engineers, 1951, Part I, Supplement 1, Central and Southern Florida Project: Agricultural and Conservation Areas - Geology and Soils.

U.S. Army Corps of Engineers, 1953, Part I, Supplement 7, Central and Southern Florida Project: Agricultural and Conservation Areas – Permeability Investigations by Well Pumping Tests.

U.S. Army Corps of Engineers, 2002, Test Pit Logs for US DOI Modified Waters to the ENP.

U.S. Army Corps of Engineers, 2009a, DECOMP Part I, L-67A Vicinity Test Pits.

U.S. Army Corps of Engineers, 2009b, DECOMP Part I, L-67A Vicinity Boring Logs.

U.S. Army Corps of Engineers, 2009c, Central and Southern Florida Project, Comprehensive Everglades Restoration Plan, L-31N (L-30) Seepage Management Pilot Project, 272 pp.

U.S. Army Corps of Engineers, 2011b, Core Borings along L-5/L-4/L-23 Waterway Water Conservation Area 3 Decompartmentalization (DECOMP) and Hydrologic Sheet Flow Enhancement Part 1Broward County, FL.

U.S. Department of Agriculture, 1996, Soil Survey of Dade County Area, Florida, 116 pp.

Wolf WPC, 2008, Report of Geotechnical Exploration L30SMPP Additional Drilling, T.O. #70, Miami, Florida.

Wolf WPC, 2009, Draft Conceptual Geotechnical Data Report, Miami Canal Decompartmentalization, Contract W912EP-05-D-0009, Miami-Dade County, Florida.

http://publications.usace.army.mil/publications/index.html for all EC's, ER's and EM's in document

A.12 ENGINEERING PLATES

Plates
Annex C-2, Civil Plates L-4 Degrade, Cross Sections L-67A, L-67C, L-29, L-67D, Miami Canal Backfill/Islands
Annex D-1, Mechanical Plates M1 – M4

A.13 ENGINEERING APPENDIX SUPPORT DOCUMENTS

The following project plans were provided as similarly designed features as referenced in the main Engineering Appendix.

NOR - FEB Features
S-623 is referenced to S-65EX1
S-624, S-625 and S-628 are referenced to HHD (S-276 (C-4A) and S-277 (C-3))
S-627 is referenced to C-111 South Dade (S-327)
S-626 PS is referenced to S-357 design and Miller (S-486) layout (structural)

SOR Features
S-620 is referenced to HHD (S-276 (C-4A) and S-277 (C-3))
S-621 and S-622 are referenced to S-65EX1
S-630 PS is referenced to S-357 design and Miller (S-486) layout (structural)
S-8A

BGY Features
S-631, S-632 and S-633 (1 barrel each) are referenced to MWD and DECOMP (S-152)
S-333N and S-355W are referenced to S-65EX1
S-356 is referenced to Miller (S-486) and Merritt (S-488) (Plates for Merritt have been provided)
L-67D is referenced to (L-67A, L-67C and L-29 levees) and DECOMP (S-152)
Barrier Wall is referenced to L-31N SMPP and L-31N Rock Wall (plans already provided)

S-65EX1 Structural, Mechanical (PROVIDED)
12R0016_Plans1.pdf through 12R0016_Plans6.pdf

S-276(C-4A) (PROVIDED)

12R0025_Plans1.pdf – General, Civil (C-3, C-4, C-7, C-10), Structural
12R0025_Plans2.pdf – Mechanical, Electrical

S-277 (C-3) (PROVIDED)

12R0025_Plans3.pdf – General, Civil (C-3, C-4, C-7, C-10), Structural
12R0025_Plans4.pdf – Mechanical, Electrical

S-327 Data005.tif (PROVIDED)

S-357 and S-488 Structural, Mechanical

357a as-builts (357m101, 357m301, M302, 357m407) (PROVIDED)
357a as-builts (all remaining) (PROVIDED)

S-486 (in design)

S-488 Site, Structural and Mechanical (M1-M4) (PROVIDED)

DECOMP S-152, 12-R-0010 (PROVIDED)

L-67A and L-67C

L-31N SMPP

09R0028_Plans.pdf (PROVIDED)
L-31N seepage barrier 2 miles construction plans 10-2011.pdf (PROVIDED)

S-346

SFWMD Structure description sheet (PROVIDED)